CORPS OF ENGINEERS, U. S. ARMY

MISSISSIPPI RIVER COMMISSION

SPILLWAY FOR BENBROOK DAM
CLEAR FORK OF THE TRINITY RIVER, TEXAS

MODEL INVESTIGATION

TECHNICAL MEMORANDUM NO. 2-269

WATERWAYS EXPERIMENT STATION

VICKSBURG, MISSISSIPPI

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SPILLWAY FOR BENBROOK DAM

CLEAR FORK OF THE TRINITY RIVER, TEXAS

Model Investigation

PART I: INTRODUCTION*

1. A comprehensive plan for flood protection and water conservation on the Trinity River and tributaries calls for modifications of existing floodways at Dallas and Fort Worth, Texas, modification of the existing Garza Dam and Reservoir, and the construction of five new reservoirs in the upper basin to be operated in conjunction with six existing reservoirs. In the interest of navigation, the plan also provides for canalization of Trinity River from the mouth to Fort Worth, with a channel across Galveston Bay to the Houston Ship Canal. Figure 1 shows the location of the proposed reservoirs and floodways involved in the over-all plan.

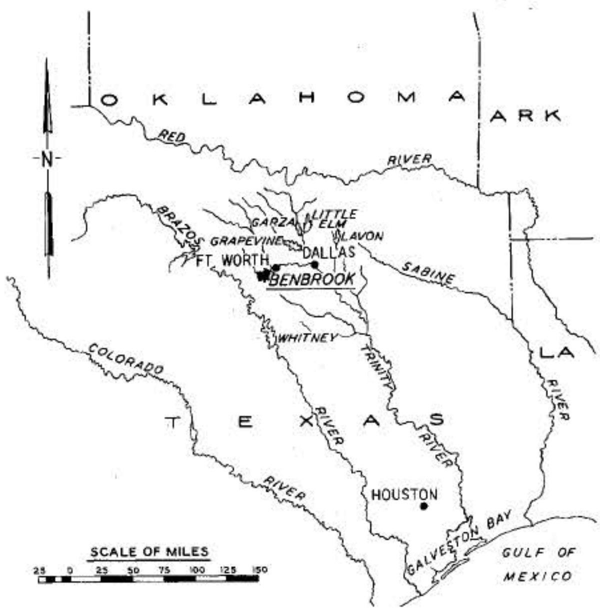

Figure 1. Vicinity map

2. Benbrook Dam, the spillway of which is the subject of this

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* Information on the prototype was obtained from maps and data furnished by the District Engineer, Galveston District, Corps of Engineers.

report, is one unit of the comprehensive plan for flood control and water conservation. The dam is proposed for construction on the Clear Fork of the Trinity River, approximately 10 miles southwest of Fort Worth, Texas. In addition to serving as one unit in the comprehensive plan for the Trinity River basin, it will, when operated in conjunction with three existing reservoirs on the West Fork of the Trinity River, provide flood protection for the city of Fort Worth. The reservoir created by the dam will have a surface area of 10,303 acres at maximum pool stage and an area of 3,769 acres at conservation pool level. A total of 322,000 acre-ft of storage will be available for flood-control purposes. The dam itself will be a rolled-fill earth embankment containing over six million cubic yards of material and will have a length of 9,200 ft and a height above stream bed of 130 ft.

3. Located in the right abutment of the dam will be an outlet works consisting of an intake structure with two slide gates 6.5 ft wide by 13 ft high, a circular conduit 13 ft in diameter and about 600 ft long, and a stilling basin (figure 2). Capacity of the outlet works at a pool elevation of 710* will be 7,340 cfs; capacity at maximum pool elevation of 741 will vary from about 8,000 cfs to 8,600 cfs, depending on tailwater effect caused by spillway discharge. To provide passage of low flows, two welded steel pipes, 30-in. outside diameter, with inlets at the intake structure will be placed parallel to the main conduit on either side and will empty into the outlet works stilling basin. Discharge through each pipe will be controlled by slide gates located in the

* All elevations are in feet above mean sea level.

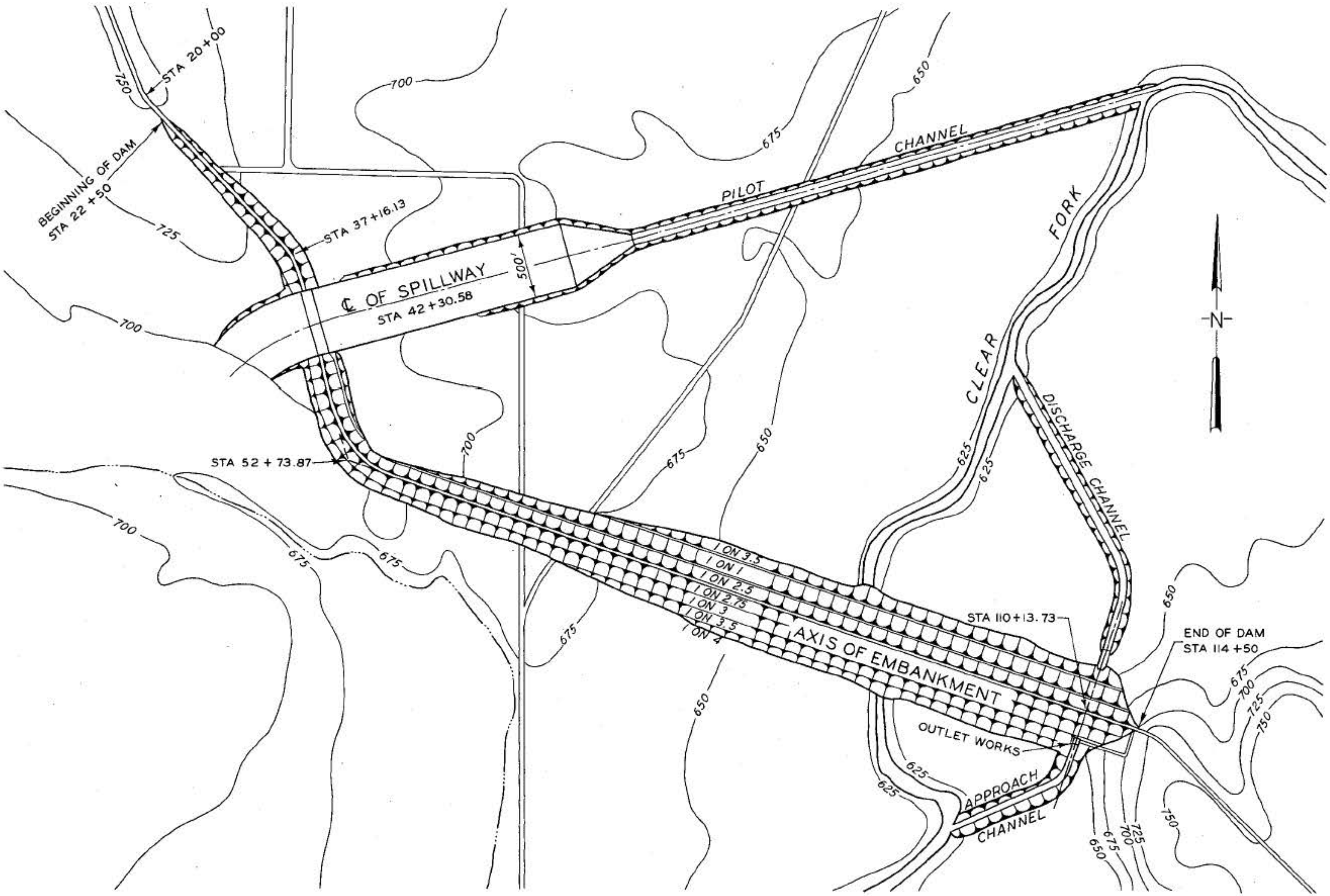


Figure 2. Proposed project plan for Benbrook Dam

intake tower. Combined capacity of the two pipes will be about 184 cfs at a pool elevation of 710 and about 196 cfs at a pool elevation of 741.

4. The spillway, located on the left bank west of the main dam, will consist of an approach channel 500 ft wide, an uncontrolled ogee crest at elevation 724 having a low section in its center at elevation 710, and an exit channel (figures 2 and 3 and plate 1). The portion of the ogee crest at the high elevation will be 400 ft in width while the portion of the crest at the low elevation will be 100 ft in width. The low-level crest is designed to pass a flow equal to the downstream channel capacity and the high-level crest is designed to pass greater flows. The two-level spillway design utilizes effectively the valuable storage area existing between the high and low spillway levels. The spillway

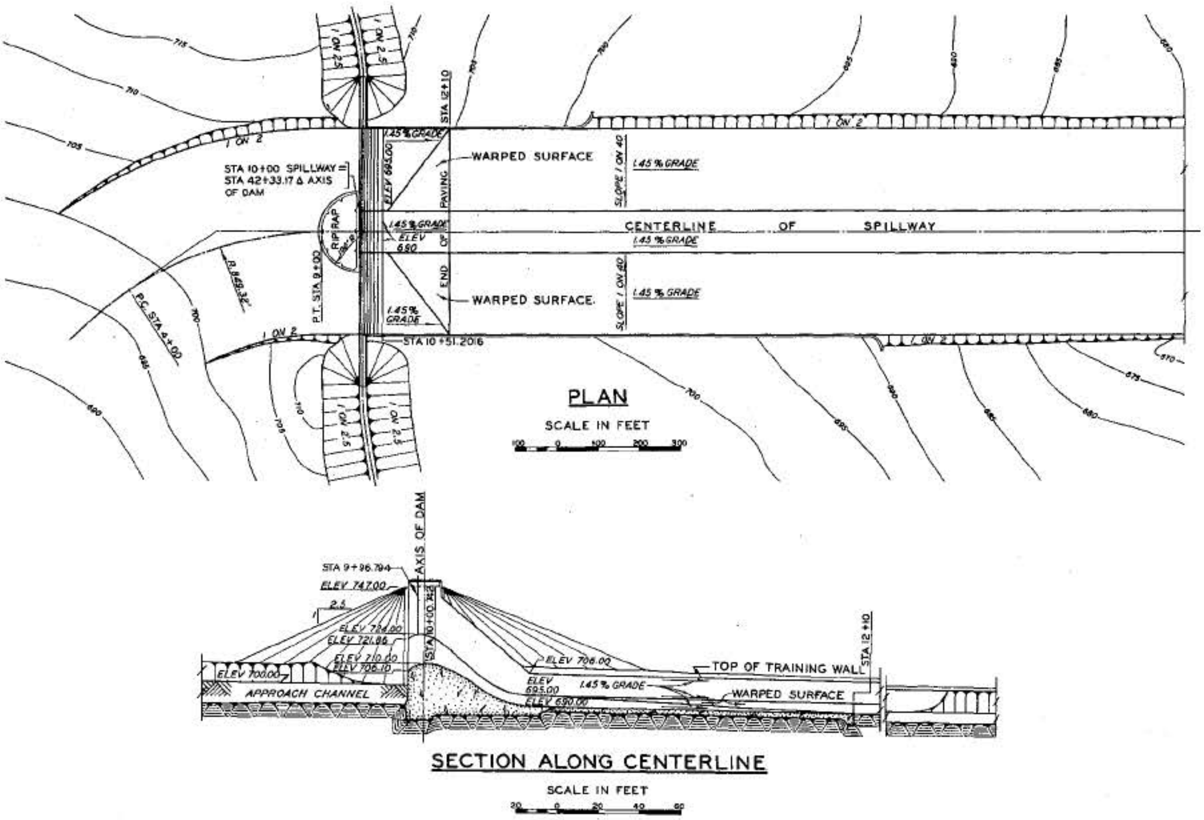


Figure 3. Spillway plan and profile

design flood, when routed through the reservoir with the pool at elevation 710 at the beginning of the flood and the outlet works closed, produced maximum heads of 31 ft on the 100-ft-wide low weir and 17 ft on the high weir with a combined discharge of 172,000 cfs. With the pool at spillway crest stage (elevation 724), the low weir section will have a capacity of 18,000 cfs.

5. The profile of the high weir, designed for a head of 17 ft, will follow the curve of $Y = 0.044987X^{1.85}$ downstream from the crest line, while upstream from the crest the profile will be shaped to arcs of circles with radii of 8.5 ft and 3.4 ft. The profile of the low portion of the spillway, designed for a head of 31 ft, will follow the curve of $Y = 0.0270X^{1.85}$ downstream from the crest, while upstream from the crest the profile will be shaped to arcs of circles with radii of 15.5 ft and 6.2 ft. The horizontal entrance curve of the low weir will be identical to the compound curve of the upstream vertical section. Height of the low and high weirs above the approach channel will be 10 ft and 24 ft, respectively.

6. The spillway will discharge onto a paved apron approximately 159 ft long, joining the exit channel at Sta 12+10. The paved section will act as a transition from the rectangular cross section at the toe of the spillway to the trapezoidal cross section of the exit channel. Concrete training walls 11 ft in height will confine the flow in the exit channel for a distance of about 500 ft downstream from the spillway. The left training wall will terminate at Sta 15+60, while the right training wall will be continued to Sta 22+60. The longer wall on the right was desired to insure the prevention of right bank erosion which would permit

spillway flow to attack the toe of the dam. A pilot channel 100 ft in width will connect the exit channel with the old river channel about 7,000 ft below the dam. It is expected that the pilot channel will ultimately scour to the full width of the exit channel and to the surface of the underlying rock. No form of energy dissipation is proposed, although a hydraulic jump is expected in the exit channel at about Sta 43+50.

7. The following data pertain to the structural and hydraulic features of the spillway:

	High Weir	Low Weir
a. Structural		
Width	400 ft	100 ft
Elevation of crest	724	710
Maximum height	24 ft	10 ft
b. Hydraulic		
Maximum head	17 ft	31 ft
Maximum pool elevation	741	741
Capacity (pool elev 724)	---	18,000 cfs
Combined capacity	172,000 cfs	

8. In order to check certain phases of the design not readily susceptible of analysis by theoretical means, a model study was deemed necessary. Specifically the purpose of the model study was to (a) verify spillway discharge coefficients, (b) investigate any objectionable flow characteristics that might result from the concentration of flow through the low portion of the spillway, and (c) determine the top grade of the right training wall to prevent overtopping. Authority to undertake the model study was granted by the Chief of Engineers in the second indorsement, dated 9 April 1947, to the basic request, dated 17 March 1947, of the District Engineer.

9. During the course of the model study, Mr. J. H. Douma of the

Office, Chief of Engineers, Mr. H. W. Feldt of the Southwestern Division, and Messrs. W. E. Wood, A. Martelli, K. V. Speeg, and G. W. Demeritt, of the Galveston District, visited the Waterways Experiment Station at intervals in an advisory capacity. The model study was conducted in the Hydraulics Division of the Waterways Experiment Station during the period May to November 1947 by Messrs. R. G. Cox and S. B. Burns, under the general supervision of Messrs. F. R. Brown and T. E. Murphy.

PART II: THE MODEL

10. The model of the spillway for Benbrook Dam was constructed in a brick flume to a linear-scale ratio of 1:60 and reproduced about 900 ft of the reservoir and approach areas, the entire spillway width, and about 2,000 ft of the exit channel (figures 4 and 5). The entire surface of the model was molded in cement mortar to sheet-metal templets. Care was exercised to obtain the proper shape of all surfaces, and those areas which were to be paved in the prototype were continuously dusted with cement and troweled until the practical limit of smoothness was attained.

11. Water used in the operation of the model was supplied by a circulating system made up of a large supply sump, numerous pumps, a constant pressure header line, and a gravity flow return line. Two venturi

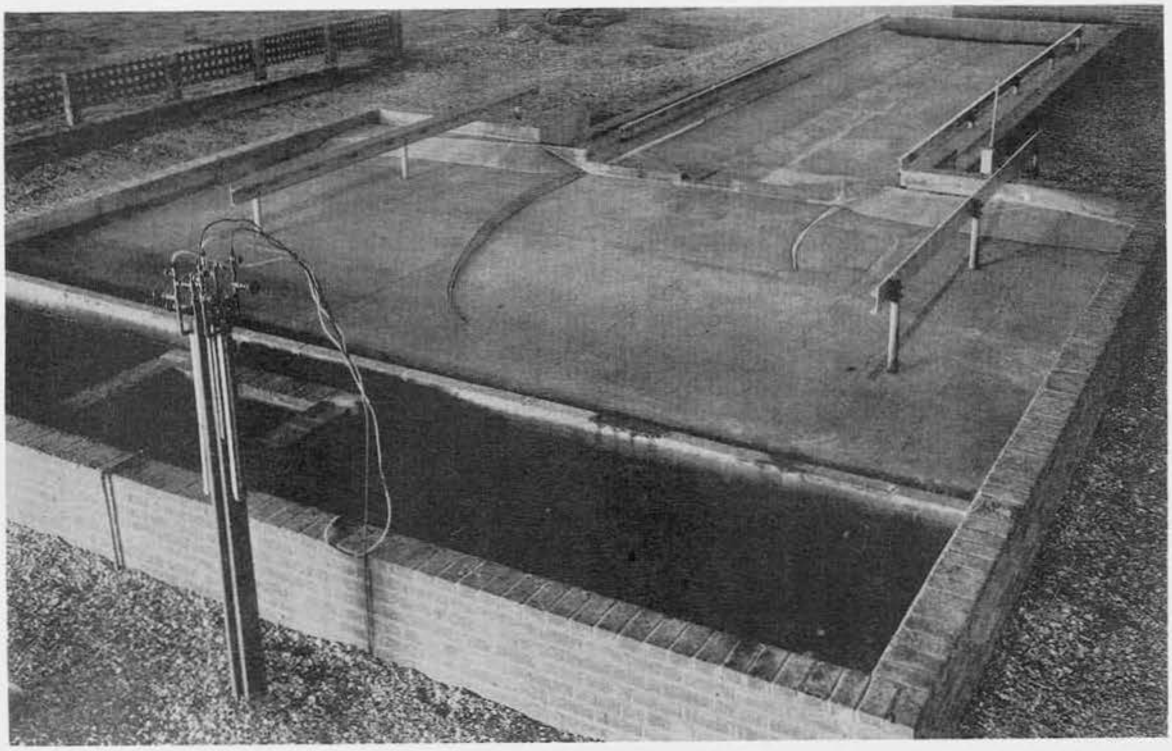


Figure 4. 1:60-scale model of Benbrook Dam spillway

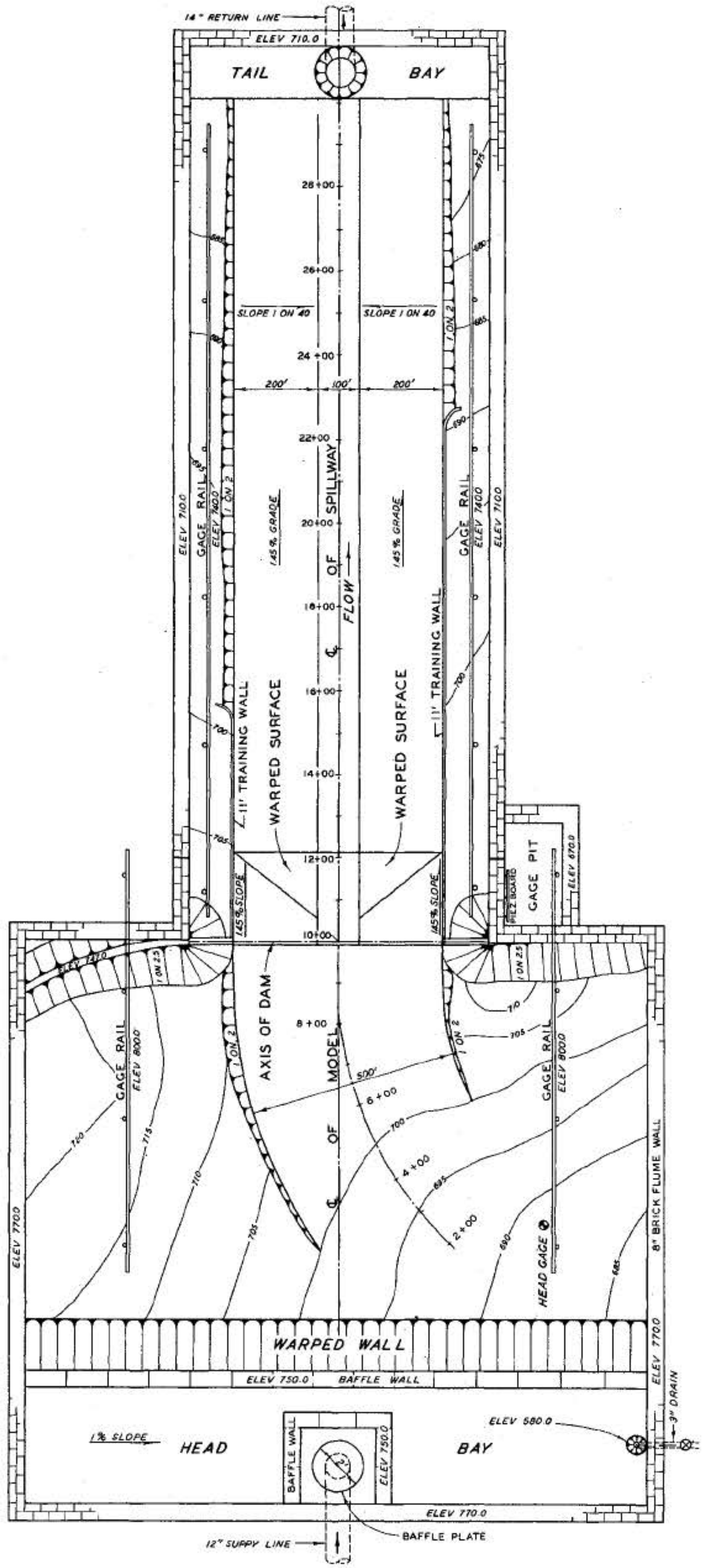


Figure 5. Model layout plan

meters located between the model and header line were used to measure the quantity of flow. From the supply lines flow spilled into a headbay where it was stilled by baffles prior to its entrance into the model. After passing through the model, the water flowed through the return line back to the sump. Steel rails, set to grade along either side of the model, provided a datum plane for the use of measuring devices. Water-surface elevations were measured both by means of portable point gages (mounted on an aluminum beam supported by the steel rails) and by means of piezometers. Velocities were measured by means of a pitot tube. Surface current directions were traced with confetti.

12. The accepted equations of hydraulic similitude, based upon the Froudian relationships, were used to express the mathematical relationships between the dimensions and hydraulic quantities of the model and the prototype. The general relationships existing for the Benbrook model are presented in the following table:

Dimension	Scale Ratio
Length	$L_r = 1:60$
Area	$A_r = L_r^2 = 1:3600$
Velocity	$V_r = L_r^{1/2} = 1:7.746$
Discharge	$Q_r = L_r^{5/2} = 1:28,886$
Roughness	$n_r = L_r^{1/6} = 1:1.989$

13. Measurements of discharge, water-surface elevations, current directions, pressures, and velocities can be transferred quantitatively from model to prototype by means of the above scale relationships. The inability of the small-scale model to reproduce assumed prototype roughness values might result in a slight decrease in spillway efficiency at

low heads. Slight variation from the theoretical model roughness should have little or no effect on spillway efficiency at high or intermediate heads or on general hydraulic performance of the spillway.

PART III: NARRATIVE OF TESTS

Types A and B (Original) Designs

14. As explained in paragraphs 4-7 the spillway for Benbrook Dam consisted of a 400-ft section with crest at elevation 724 and a 100-ft section located in the center with crest at elevation 710 (figure 6). Design heads for the two sections were 17 and 31 ft, respectively. In order to align the vertical upstream faces of the two spillway sections, the crest of the high weir was located at Sta 9+96.79, while that of the low weir was a slight distance downstream at Sta 10+00.74 (figure 3 on page 4 and plate 1). The abutments of both weirs were originally installed in the model with right angle corners (type A) and were

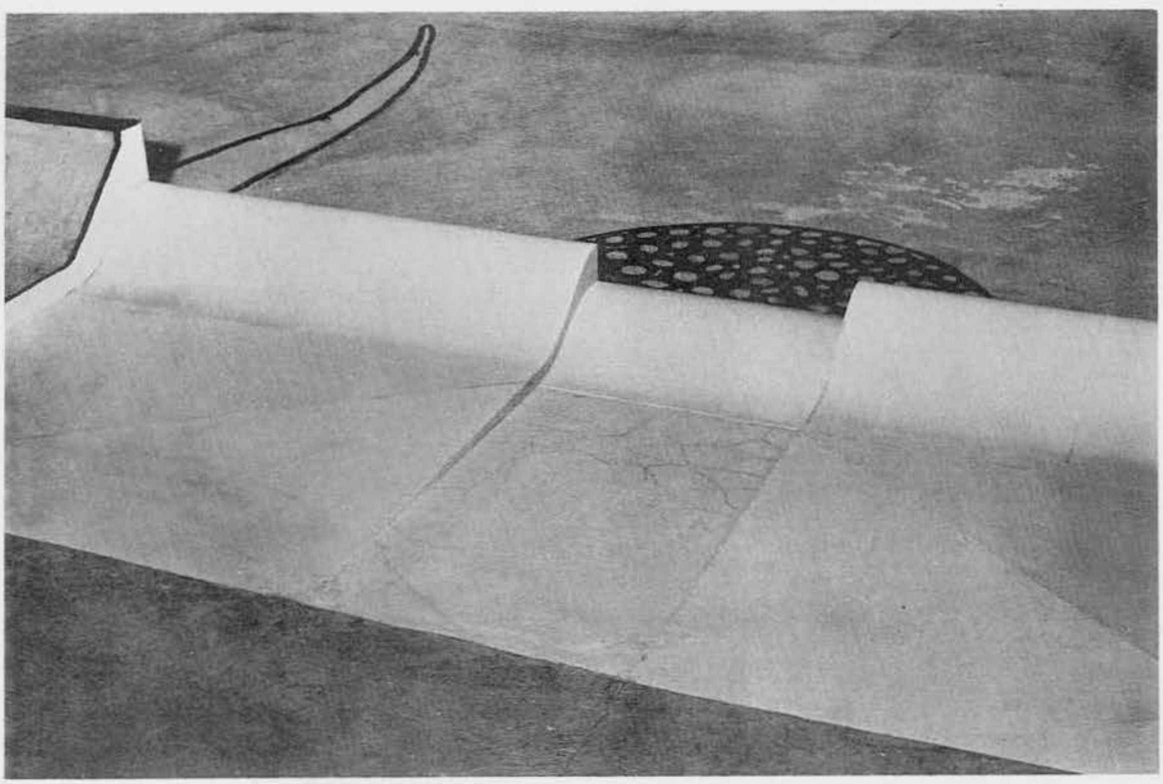


Figure 6. Upstream view of original two-level spillway design

subsequently rounded (type B) to conform to the same compound curve as the upstream portion of the crest shape. The depressed center section of the spillway extended for about 200 ft downstream until it joined the exit channel. The exit channel was sloped on a 1.45 per cent grade in the direction of flow, and in cross section consisted of a 100-ft-wide level channel located in the center with sides extended on a 1-on-40 slope to an intersection with the side training walls. The slight depression in the center of the exit channel was desired to confine low spillway discharges to the center of the channel. Training walls on either side of the spillway extended to Sta 15+60 on the left and to Sta 22+60 on the right.

15. Measurement of head-discharge relations of flow over the spillway crest revealed that model and computed results were in close agreement throughout the range in discharge (plate 2). The design discharge of 172,000 cfs was passed at a pool elevation of 741.2 as compared to a computed elevation of 741.0. For the design discharge through the low portion of the spillway (18,000 cfs) there was no detectable difference between model and computed results. Computations were based on the formula $Q = CL'H^{3/2}$, where

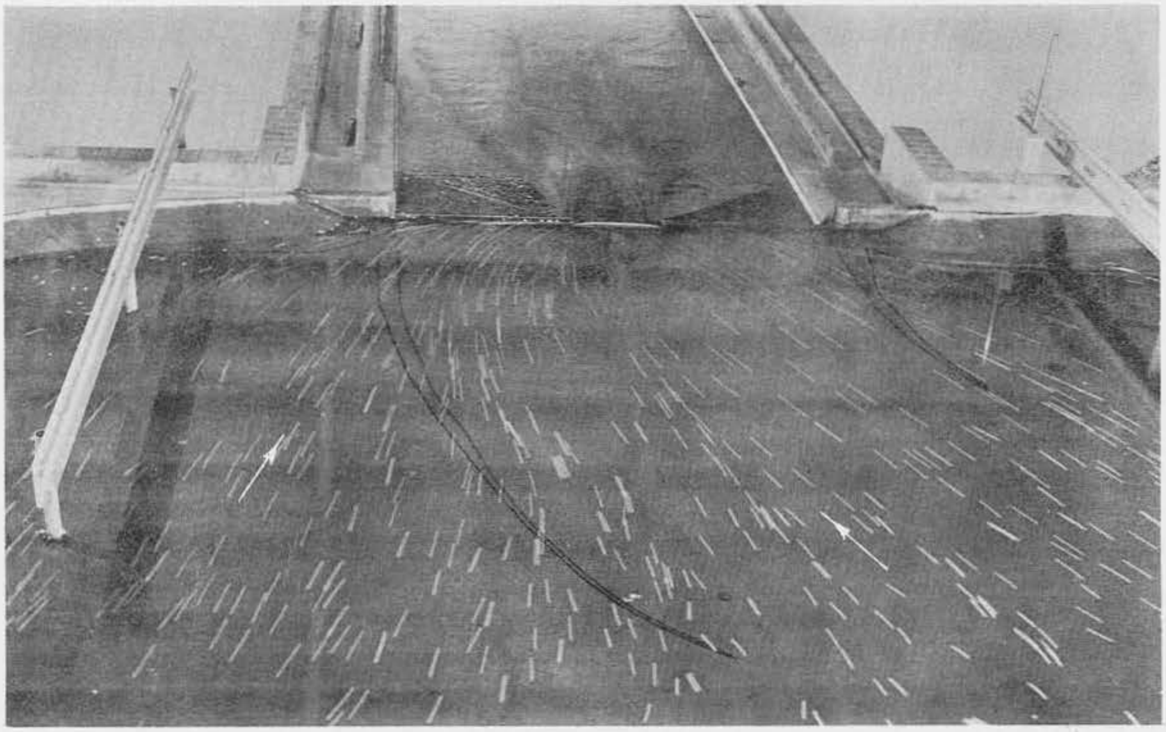
Q = discharge in cfs

C = coefficient of discharge which varied from 3.09 at zero head to 3.90 at the design head for the high weir section and from 3.09 to 3.70 for the low weir section

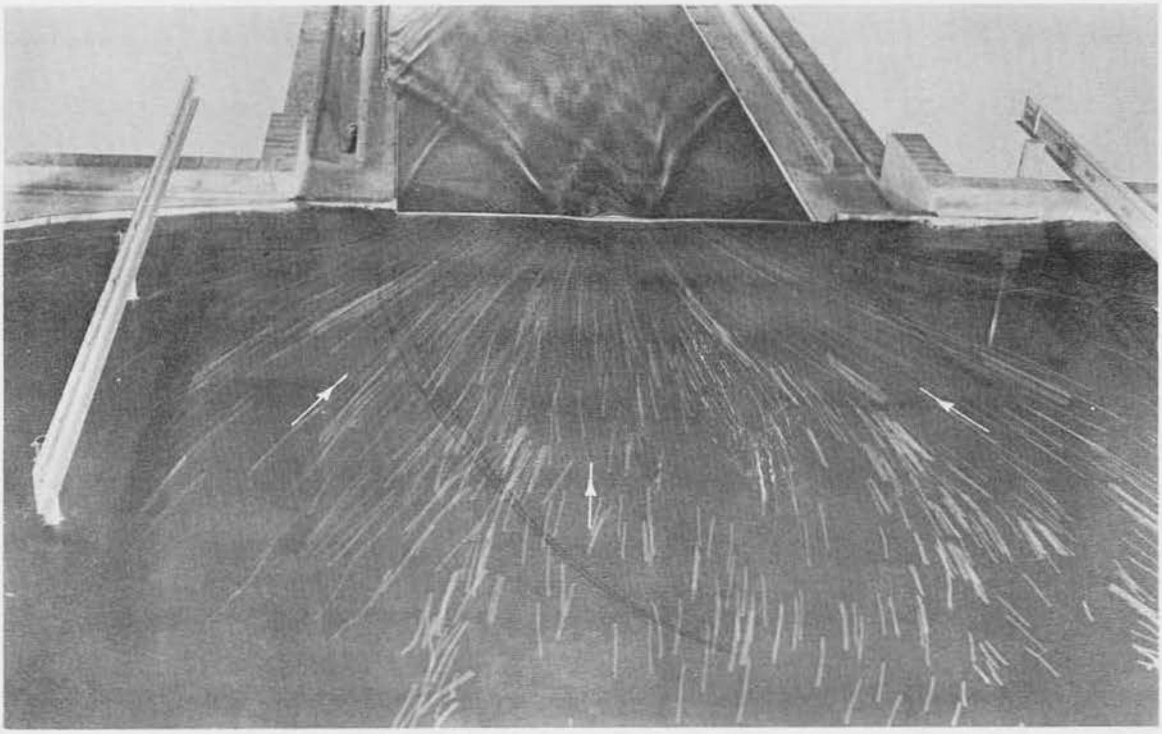
L' = effective length = $L - 0.04H$

H = head on each particular weir section

16. Flow conditions in the approach channel upstream from the weir were generally satisfactory for discharges up to about 100,000 cfs



a. Discharge 18,000 cfs; pool elevation 724.0 ft

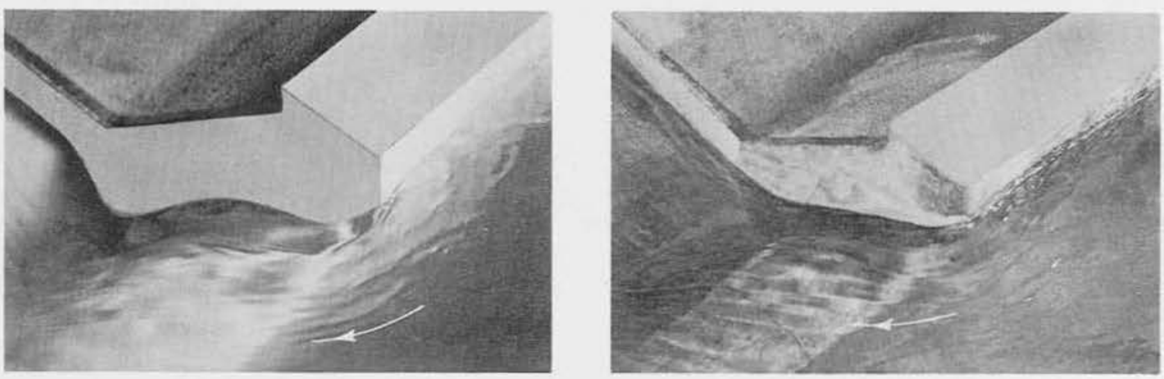


b. Discharge 172,000 cfs; pool elevation 741.2 ft

Figure 7. Flow patterns in the approach channel of original design for two discharges (confetti streaks show direction and relative velocity of surface currents)

(figure 7a). For the maximum discharge of 172,000 cfs velocities were as high as 11 ft per sec upstream from the low weir and 9 ft per sec along the rolled-fill earth embankment adjacent to the spillway (figure 7b). Plates 3-6 present approach channel velocities for the selected discharges.

17. With the exception of conditions at the abutments of both weirs, flow over the spillway was satisfactory. At the abutments of each of the two weir sections it was observed that, unless the corners were rounded, considerable turbulence resulted (figure 8). The rounding of the corners to the same curvature as the upstream portion of the high weir caused flow to follow more nearly the wall alignment, resulting in smoother flow conditions. Pressure measurements procured on the side walls of the low weir are also indicative of the improved performance effected by rounding the corners to provide a smoother transition of flow (plate 7). With the originally-installed sharp corners (type A) and a maximum discharge of 172,000 cfs, negative pressures on the side of the low weir section adjacent to the upstream face were as high as -14 ft of water. The rounding of the corners reduced the magnitude of the negative



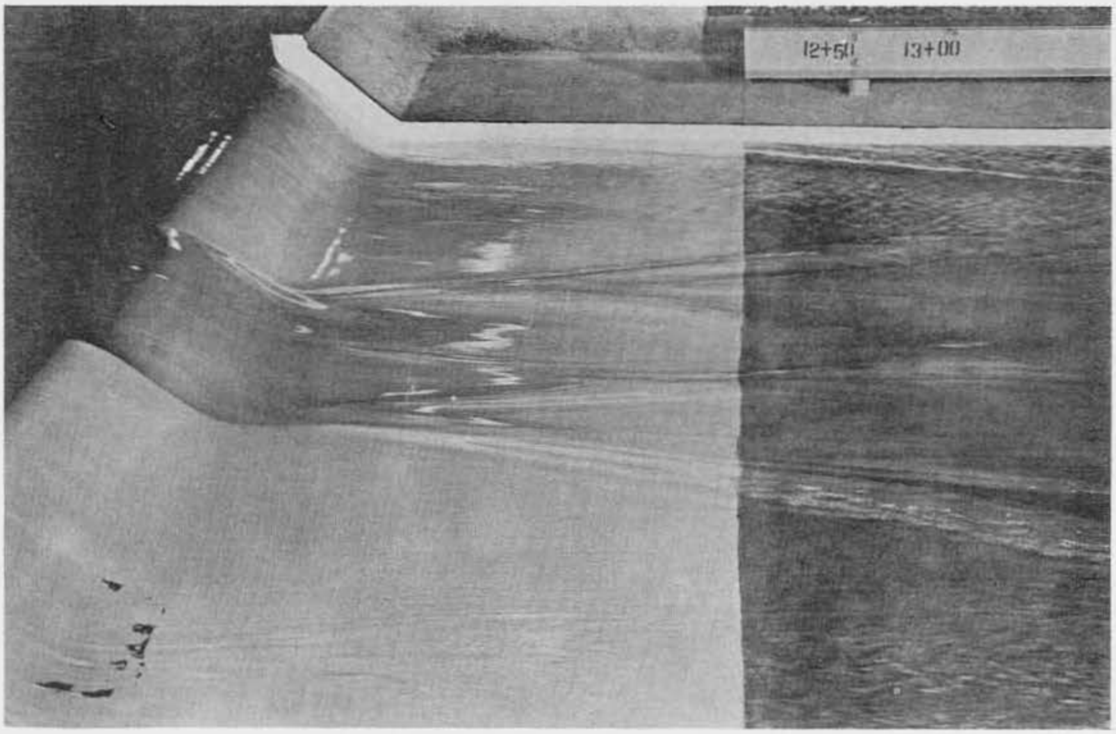
Sharp cornered abutment (type A) Rounded abutment (type B)

Figure 8. Flow around right spillway abutment; discharge 172,000 cfs

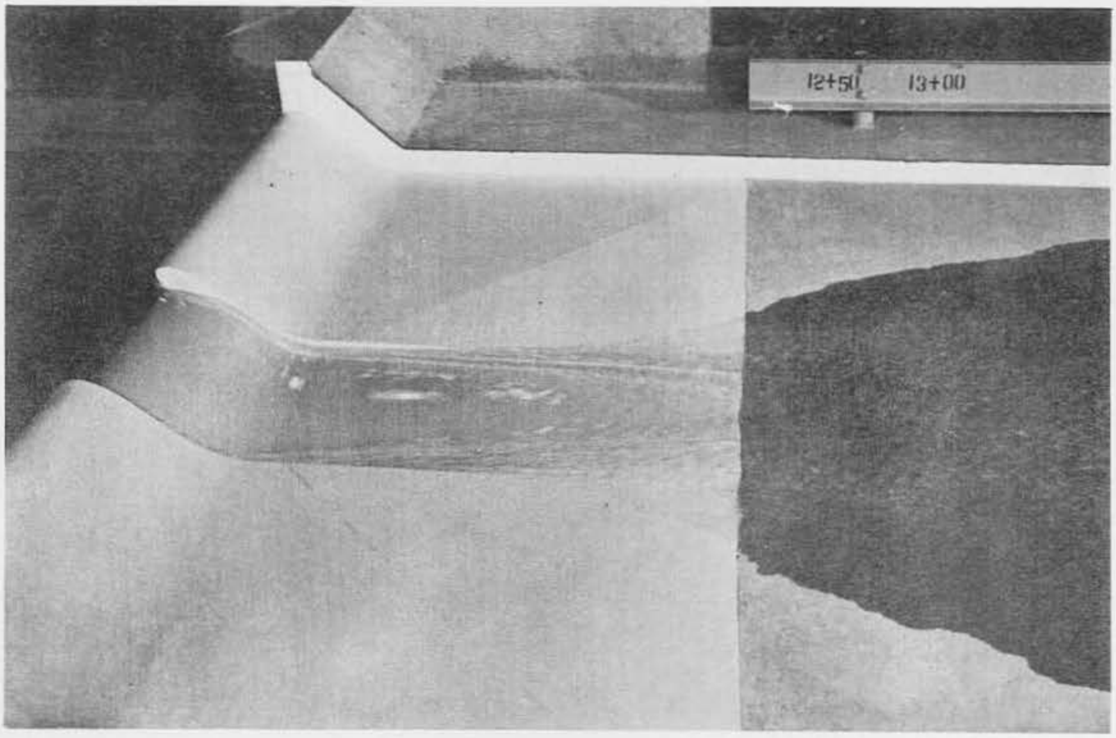
pressure to -5 ft of water. No pressures were measured on the side training walls of the high weir.

18. Flow conditions and water-surface profiles over the spillway and through the exit channel for discharges of 172,000, 100,000, and 18,000 cfs are shown on figures 9-11 and plates 8-11. Concentration of flow over the low weir section in the center of the spillway resulted in two diverging waves in the exit channel for all discharges. Increased discharge had little effect on the pattern of the waves, but did increase the height. For discharges in excess of 100,000 cfs the energy of flow was such as to cause the diverging waves to extend entirely across the exit channel and attack the side banks at about Sta 16+00. At the right bank where the wave encountered the training wall some overtopping of the wall resulted. On the left bank the wave extended downstream beyond the end of the training wall at the maximum discharge and impinged directly on the 1-on-2 side slope of the channel. In general, the standing waves began at about Sta 10+06 at the downstream edge of the low weir and terminated at about Sta 16+00 on the sides of the channel. The water-surface profiles presented indicate the conformation of the waves as they continued downstream from the spillway. Although the profiles show sufficient freeboard on the training walls for the maximum discharge of 172,000 cfs, wave action caused some overtopping of the right wall at about Sta 16+00. Overtopping of the right wall creates an undesirable condition in that drainage from the area at the right wall is directly toward the toe of the dam.

19. As mentioned previously in the report, no form of energy dissipation was planned below the spillway. As a result, flow from the

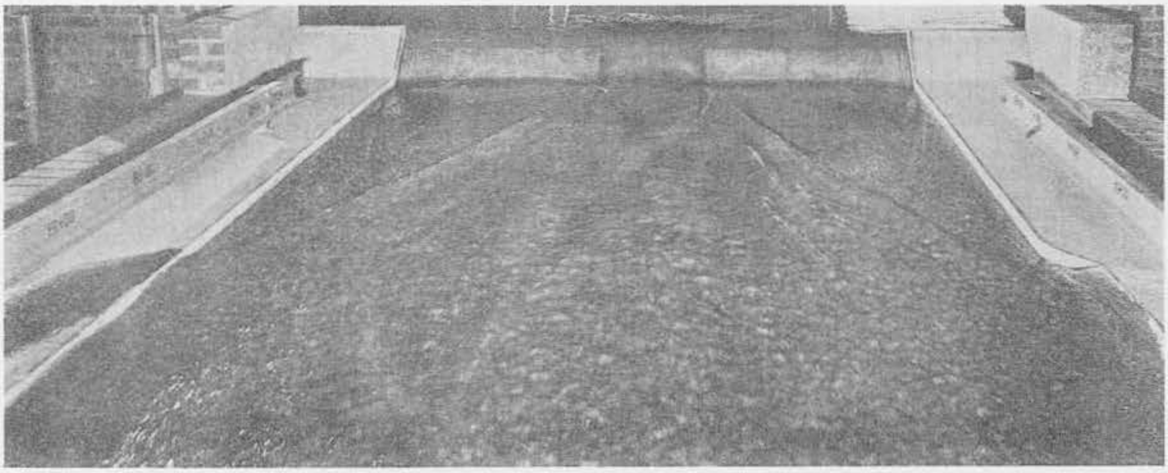


a. Discharge 172,000 cfs; pool elevation 741.2 ft

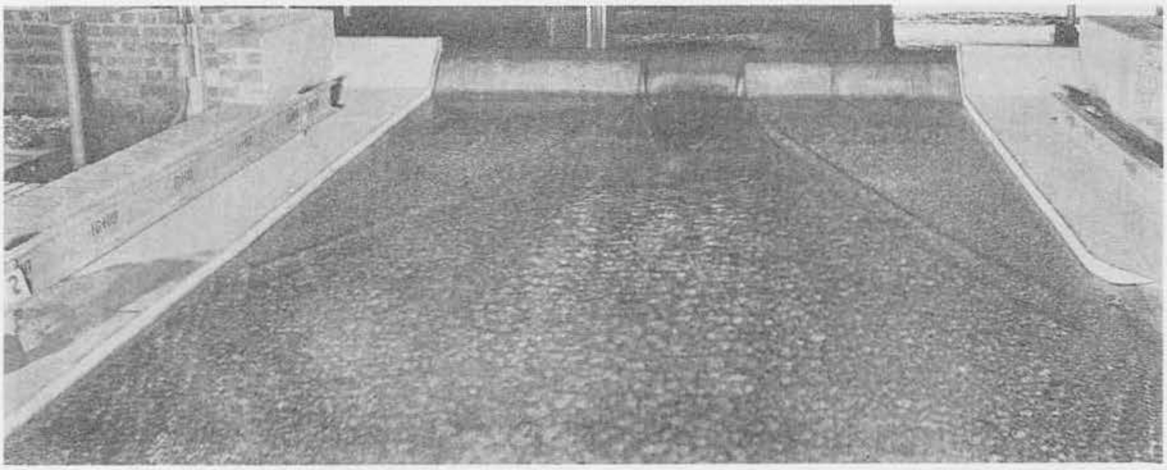


b. Discharge 18,000 cfs; pool elevation 724.0 ft

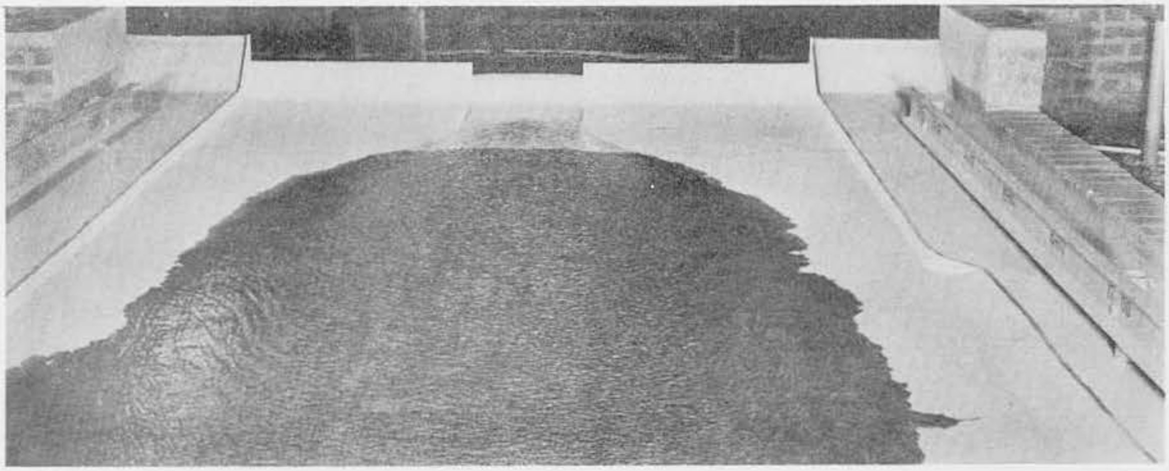
Figure 9. Flow conditions over original design spillway for two discharges



Discharge 172,000 cfs

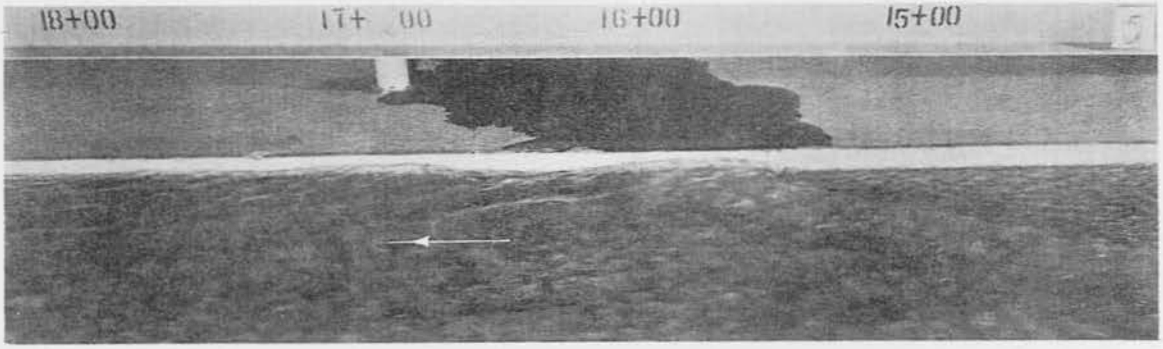


Discharge 100,000 cfs

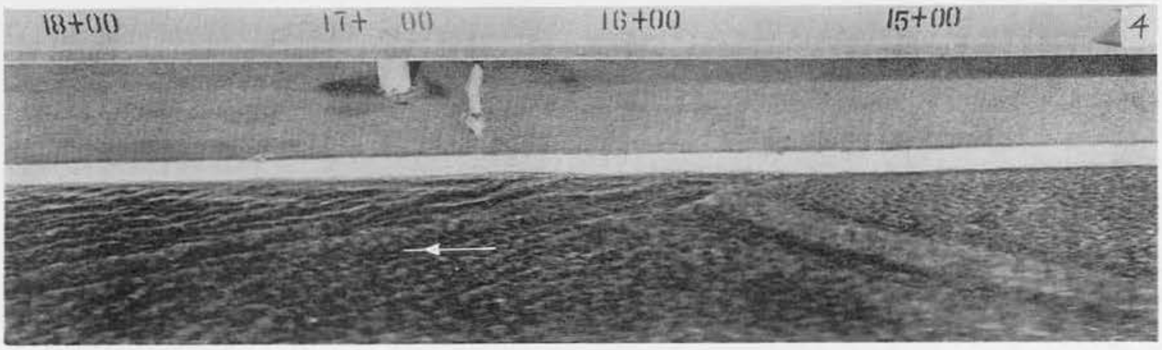


Discharge 18,000 cfs

Figure 10. Flow conditions in original design exit channel for three discharges



Discharge 172,000 cfs



Discharge 100,000 cfs

Figure 11. Wave action against right exit channel wall of original design for two discharges

spillway continued unchecked through the exit channel at sub-critical depths. Velocities recorded ranged from 25 to 50 ft per sec at the maximum discharge to 19 to 45 ft per sec at a discharge of 18,000 cfs (plates 3-5). Goodland limestone material will form the floor of the exit channel from Sta 12+10 to 50+00. It is believed by the District Office that the erosional resistance of this material is such that paving of the exit channel will not be required.

Alternate Designs

20. In an attempt to eliminate or reduce the standing waves

resulting from concentration of flow over the low weir during high discharges, numerous alterations to the spillway and area immediately downstream were investigated. Although it was believed that a smooth transition of flow could be effected from the spillway to the exit channel by a deeper channel through the center of the exit area, the District Office held that such a channel would not be economically justified. Consequently, efforts to improve conditions were confined for the most part to the spillway proper. Initial efforts were directed toward securing more uniform flow distribution in the exit area by increasing water-surface elevations below the high weir to parallel flow issuing from the low weir section (types C and D designs, plates 12 and 13). A similar attempt was made to lower water-surface elevations over the low weir to agree with those over the high weir by moving the crest line of the low weir upstream (type G design, plate 14).

21. Test results revealed similar flow conditions for all alterations investigated. The diverging wave below the low weir appeared to be unchanged in height, although the point of intersection of the wave and the training wall moved slightly upstream or downstream from its original position. An increase in the level of the downstream portion of the high weir raised water-surface elevations as anticipated but in so doing also raised water-surface elevations below the low weir an equal amount (plate 15). Relocation of the low weir 3.95 ft upstream from its original position lowered water-surface elevations slightly, but inasmuch as the distribution of flow across the spillway was unchanged the diverging waves were still present as in the original design (figure 12). Alterations to the warped concrete apron immediately below

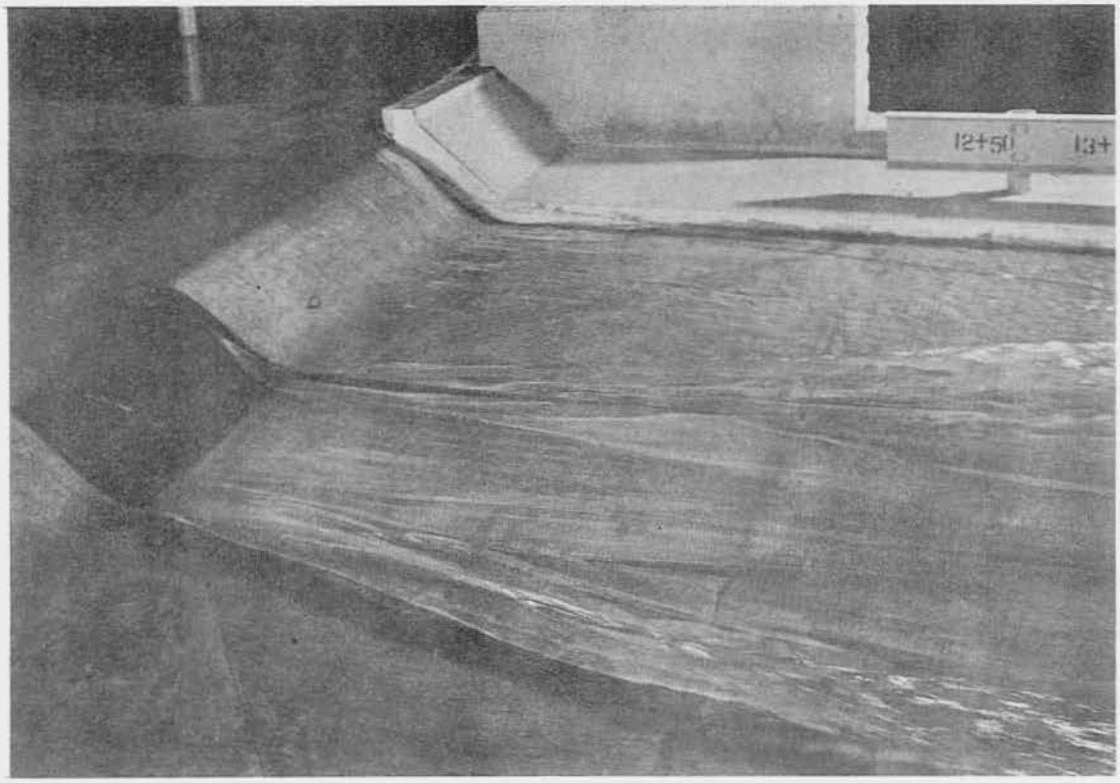


Figure 12. Discharge 172,000 cfs; low weir moved 3.95 ft upstream, type G

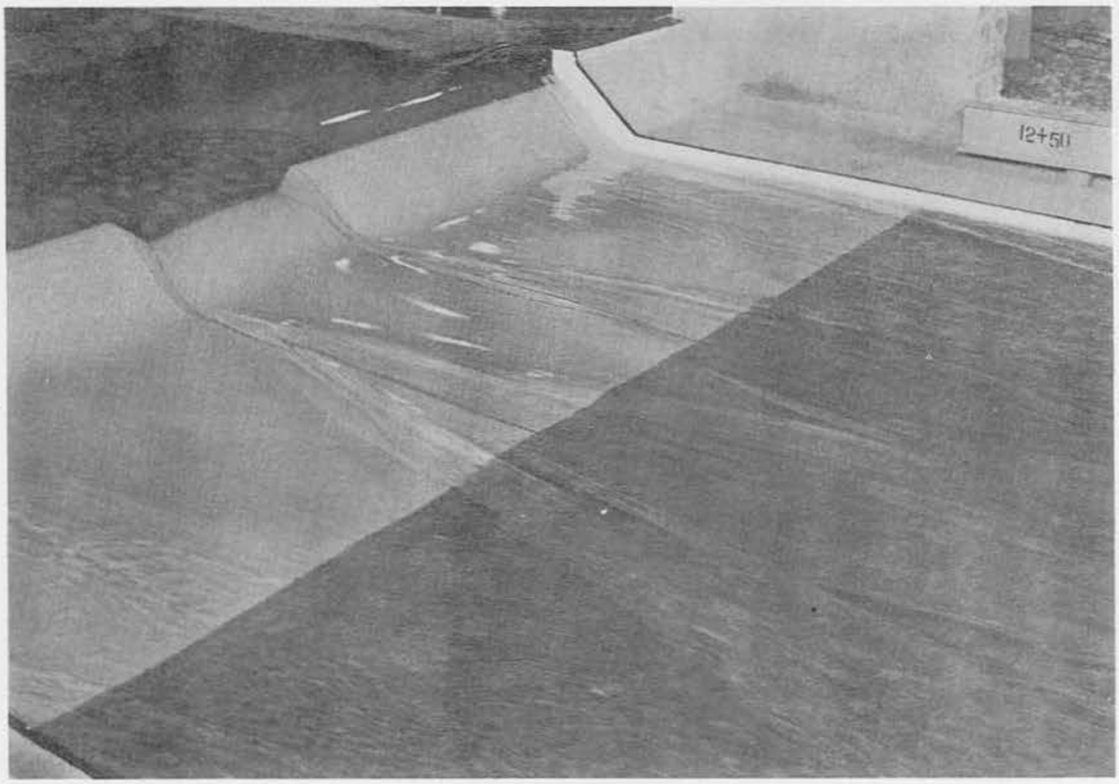


Figure 13. Discharge 172,000 cfs; low weir with 1-on-1 side slopes, type I

the spillway or variations in the side slopes of the low weir (type I) were all unsuccessful (figure 13).

22. In one series of tests the width of the low weir was varied from 100 ft to a minimum of 22 ft. It was found that as the width of the low weir was reduced the height of the wave in the exit channel decreased. For example, at Sta 12+00 the wave height was decreased from 6 ft for a width of 100 ft to 3.5 ft for a width of 22 ft. However, sufficient improvement was not obtained to warrant the decrease in spillway capacity.

23. Since no solution to the problem appeared evident with a single low weir section located in the center of the spillway, efforts were directed toward improvements by relocation of the low weir. For one test the entire low weir was moved adjacent to the right abutment (type H) while in another test a low weir 50 ft in width was located 50 ft to the left of the right abutment (type J). If it appeared feasible to relocate the low weir, it was thought that perhaps two 50-ft sections located near the abutments might be satisfactory. However, in each case, resulting flow conditions were unsatisfactory (figures 14 and 15). The wave from the right side of the low weir overtopped the right training wall, while the wave from the left side of the low weir was similar to that observed with other designs installed.

Spillway Abutments

24. As explained previously the spillway was connected to the rolled-fill earthen embankment on each side by concrete nonoverflow sections approximately 125 ft in length. It was the opinion of the District

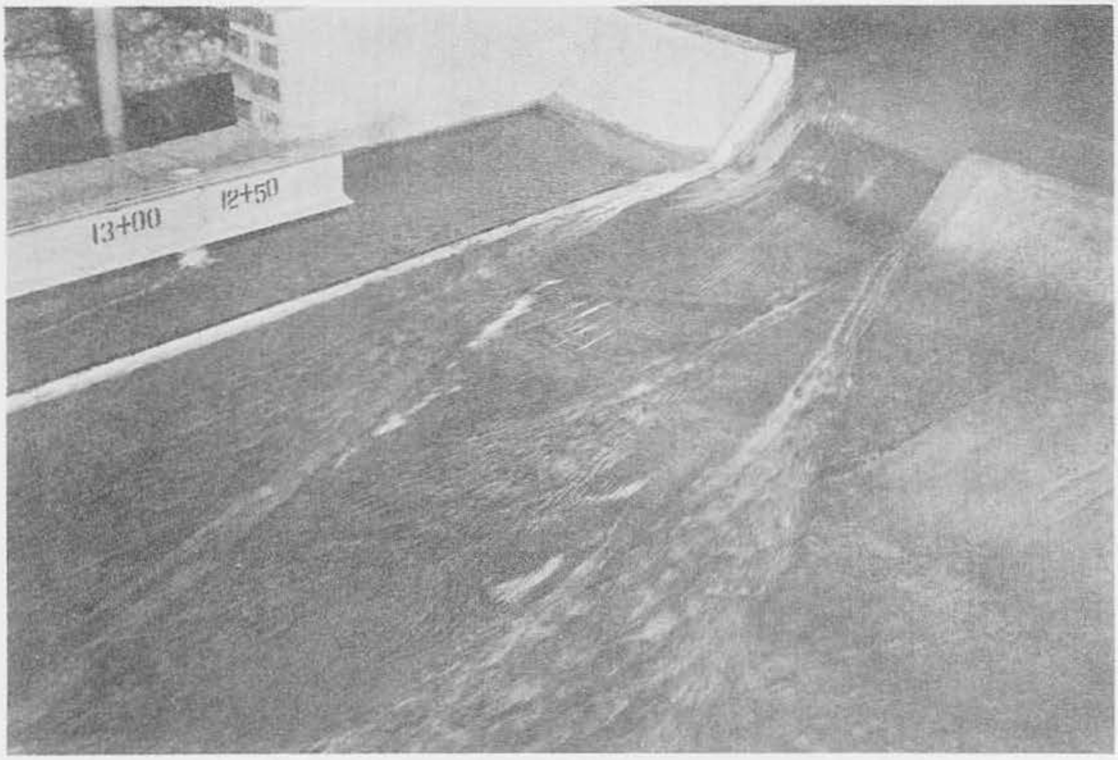


Figure 14. Exit channel flow with 100-ft low weir at right abutment; discharge 172,000 cfs, type H

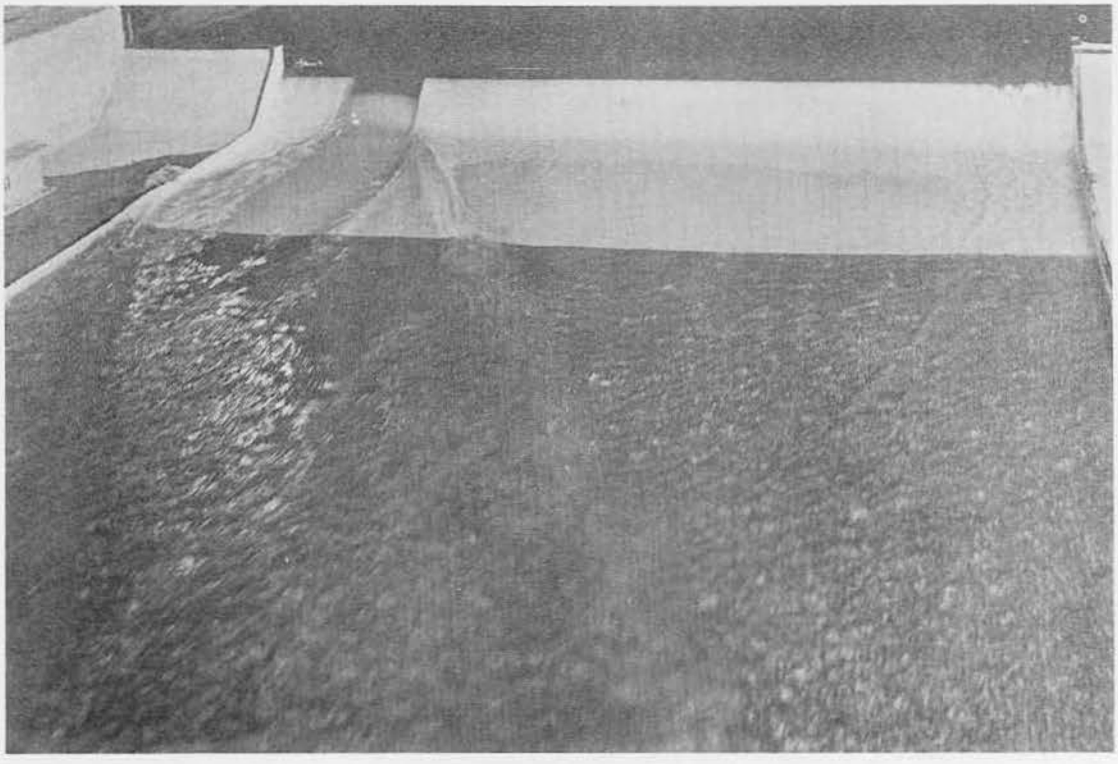
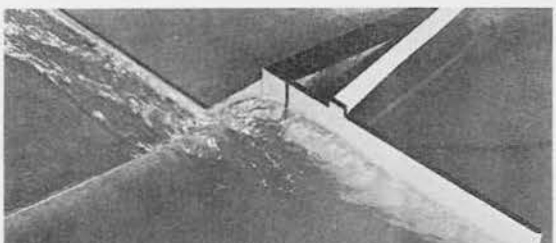


Figure 15. Exit channel flow with 50-ft low weir fifty feet north of right abutment; discharge 172,000 cfs, type J

Office that some monetary saving could be effected by extending the earth embankment sections to the spillway, thus eliminating the concrete sections. This procedure would require training walls at each abutment to protect the embankment sections from erosion resulting from spillway discharges. To expedite testing a separate type of training wall was installed at each abutment, thus permitting the investigation of two type walls simultaneously. Unless considerable improvement in hydraulic conditions could be effected, the maximum length of training wall that could be economically justified was about 30 ft.

25. At the right abutment the use of a vertical training wall with a top elevation of 747 and a length of 120 ft was investigated. The length of wall was successively reduced from 120 ft to 60, 30, and 15 ft. In addition, one test was made with the 60-ft wall at an angle of 45 degrees to the axis of the dam. For all tests the maximum discharge was used. No actual change was made in the location of the earth embankment in the model. Test results revealed that, although the use of a training wall reduced currents in the area to the rear of the wall, considerable turbulence existed on the spillway side of the wall



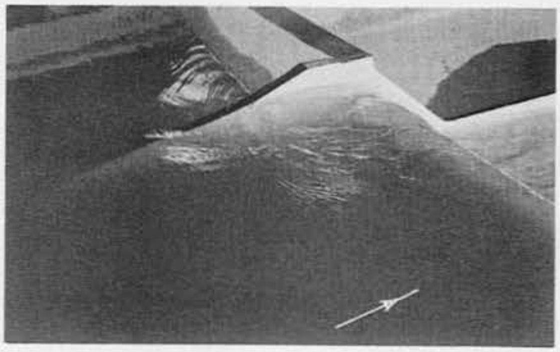
120-ft wall

15-ft wall

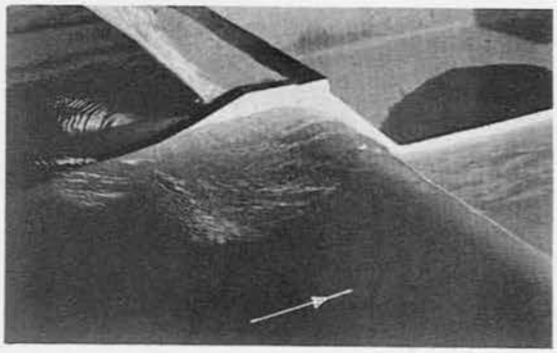
Figure 16. Flow conditions at spillway with vertical training wall at right abutment; discharge 172,000 cfs

(figure 16). As the length of the wall was decreased, conditions on the spillway side improved but velocities behind the wall increased. Location of the wall at an angle of 45 degrees to the axis of the dam resulted in less favorable conditions than those existing with walls paralleling the approach channel.

26. At the left abutment a sloped training wall paralleling the channel was investigated. The wall extended upstream for a distance of about 130 ft and sloped from elevation 747 at the spillway to the elevation of the approach channel. The earthen embankment was continued to the rear of the wall. Tests indicated considerable turbulence at the junction of the flow along the earthen embankment and the high-velocity flow approaching the spillway. Velocities recorded over the embankment section adjacent to the wall ranged from 6 to 9 ft per sec (plate 16). In an attempt to reduce velocities over the embankment section the wall was extended above the elevation of the embankment in successive tests by 3, 5, and 7 ft. It was observed that, although a slight reduction in velocity was effected by use of a 3-ft parapet wall, succeeding increases in height had little effect (plates 16 and 17). However, each increase

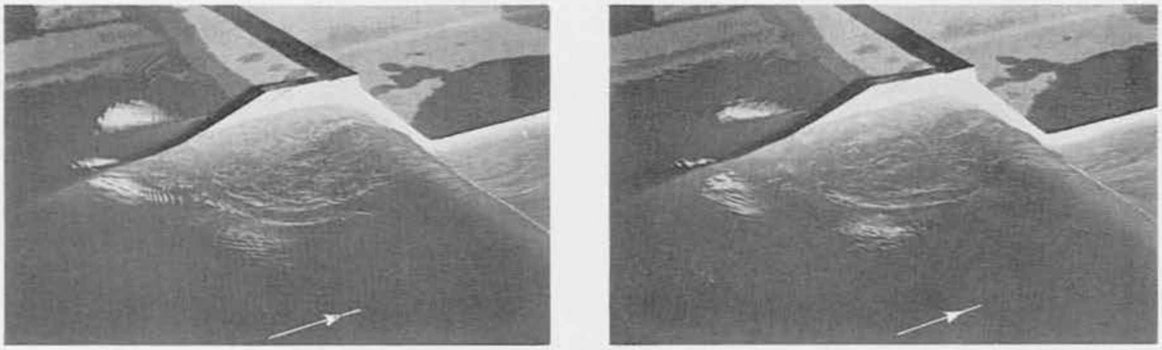


Wall with no parapet



Wall with 3-ft parapet

Figure 17. Flow conditions at spillway with training wall and with and without parapet installed at left abutment; discharge 172,000 cfs



Wall with 5-ft parapet Wall with 7-ft parapet

Figure 18. Flow conditions at spillway with training walls and two heights of parapet installed at left abutment; discharge 172,000 cfs

in height of wall increased the area of turbulence adjacent to the left training wall (figures 17 and 18). Comparison of flow conditions at the spillway abutments, with the earthen embankment extended, with flow conditions observed with the original design in place (figures 8 on page 15, and 17 and 18) reveals that the use of the rounded abutment of original design resulted in improved flow conditions over the spillway.

PART IV: DISCUSSION OF RESULTS

27. The test results described in the preceding paragraphs suggested the retention of the original spillway design with a few revisions to the training walls. While the original design is not ideal hydraulically, the benefits to be obtained, both hydrologically and economically, are judged by the Galveston District to be sufficient to justify the sacrifice of hydraulic perfection. Although improved conditions could have been secured by the use of a constant-level spillway or the excavation of a deeper exit channel immediately below the low weir section, these revisions were not considered necessary. The two-level spillway was desired because the low weir permitted flood releases without exceeding the capacity of the Fort Worth floodway downstream while the high weir provided maximum storage for flood control. Excavation of the exit channel immediately below the low weir was not permitted, since extra rock excavation would be involved resulting in a more costly structure. Although not investigated, it is believed that more favorable exit channel conditions could have been obtained by a deeper central channel which would have permitted an easier transition of flow from the low weir to the full width of the exit channel. Consideration also was given to the use of a considerable number of small openings, but this idea was abandoned because of extra construction costs and the possibility of the small openings collecting debris.

28. Tests of the Benbrook spillway indicated that bottom velocities in the approach channel immediately upstream from the low weir section and over the embankment along the upstream face of the nonoverflow

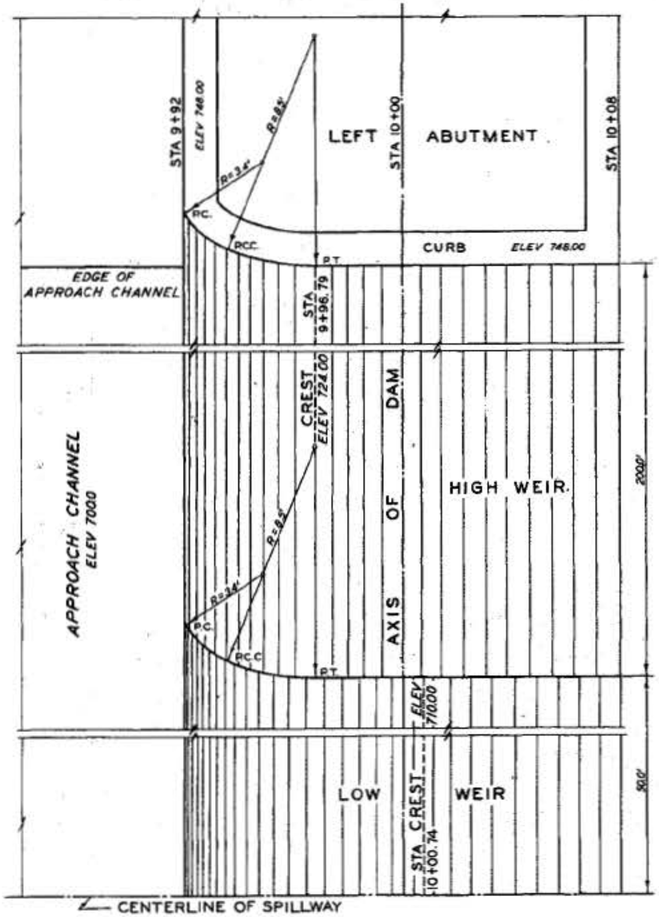
sections were in the range of 11 to 9 ft per sec, respectively. These velocities are sufficient to warrant the use of riprap to insure protection from erosion. Velocities in the approach channel upstream from the high-level weir were about 6 ft per sec. Attempts to effect some monetary saving by extending the earth embankment sections to the spillway, thus eliminating the concrete nonoverflow sections, were unsuccessful. The training walls necessary at each abutment to protect the earth embankment section would be more costly than the originally-proposed concrete sections.

29. The close agreement between the spillway rating curve computed by the Galveston District and that developed during the model study indicates the reliability of assumed discharge coefficients. The rounding of the abutments of the high and low weir sections provided improved flow conditions over those observed with square corners. A considerable reduction in the negative pressures on the side walls of the low weir section also was effected by use of the rounded walls.

30. Discharges over the low weir section at pool elevations below 724 were satisfactory. Flow was confined for the most part to the center of the exit channel and where discharges were sufficient to spread to the full channel width highest velocities were recorded in the center of the channel. Velocities in the exit area for a capacity discharge of 18,000 cfs ranged from 19 to 45 ft per sec. However, at the maximum design discharge of 172,000 cfs over the entire spillway width flow conditions were not good. The concentration of flow in the center of the spillway resulted in the formation of high standing waves which diverged from the side walls of the low weir section to the full width of the

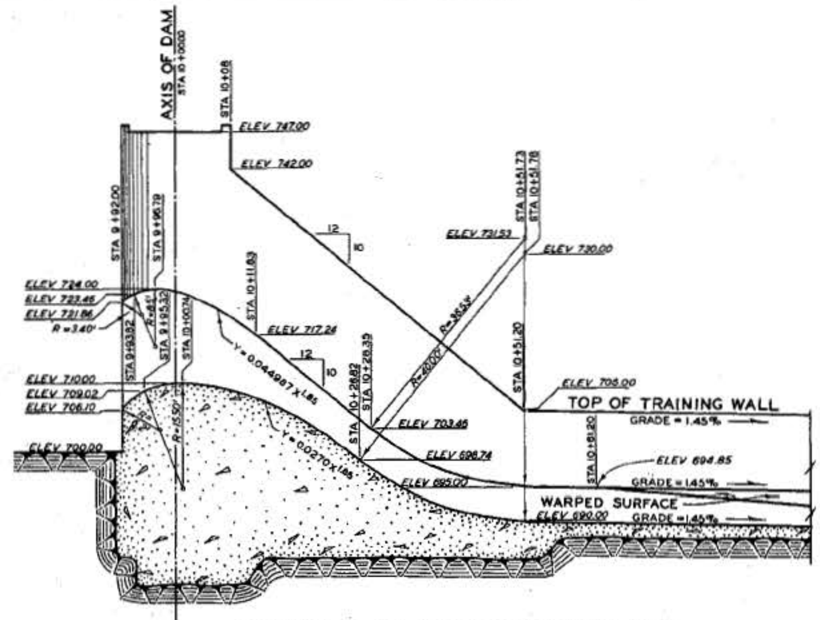
exit channel about 525 ft downstream. Where the wave struck the vertical wall on the right side of the channel, considerable flow splashed over the top of the wall; at the left side no overtopping of the bank was noted. To insure protection of the banks from the wave action noted above, the left training wall should be extended about 140 ft to Sta 17+00 and riprap placed behind each of the walls at about Sta 16+00. For discharges of 100,000 cfs or below no overtopping of the training walls by wave action was observed. Bottom velocities in the exit area ranged from 25 to 50 ft per sec for the maximum discharge of 172,000 cfs and from 20 to 50 ft per sec for a discharge of 100,000 cfs. Velocities in the center of the channel were from 5 to 10 ft per sec more than along the side walls at the maximum discharge, while velocities were about 10 to 15 ft per sec more in the center for a discharge of 100,000 cfs.

PLATES



HALF PLAN

SCALE IN FEET



SECTION ALONG CENTERLINE

SCALE IN FEET

NOTE: ORIGINAL DESIGN DESIGNATED TYPE A AND TYPE B

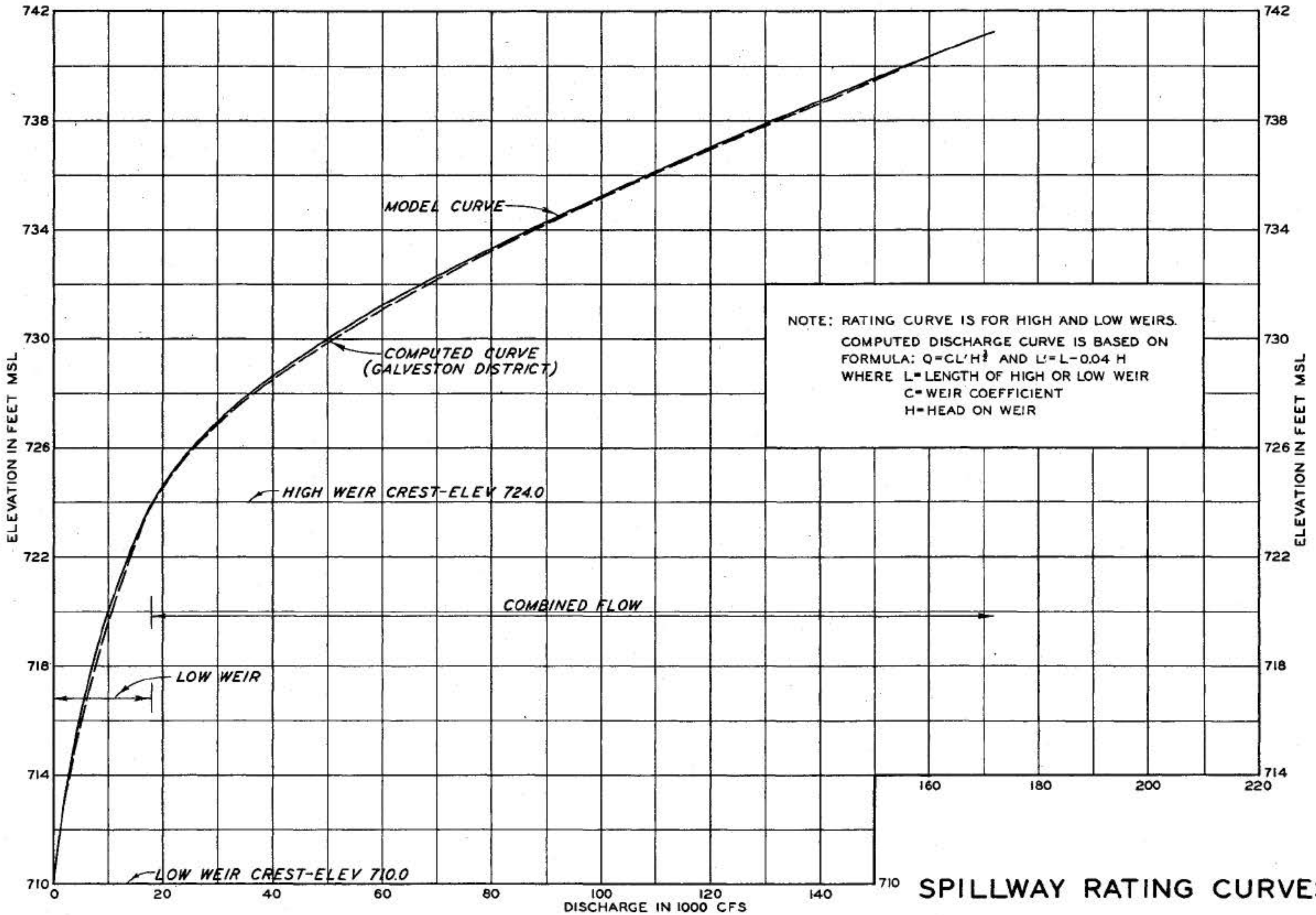
WHERE: 1. TYPE A CONSISTED OF ORIGINAL DESIGN WITH ABUTMENTS AND LOW WEIR HAVING 90-DEGREE CORNERS.

2. TYPE B CONSISTED OF ORIGINAL DESIGN WITH ABUTMENTS AND LOW WEIR ROUNDED AS SHOWN ABOVE.

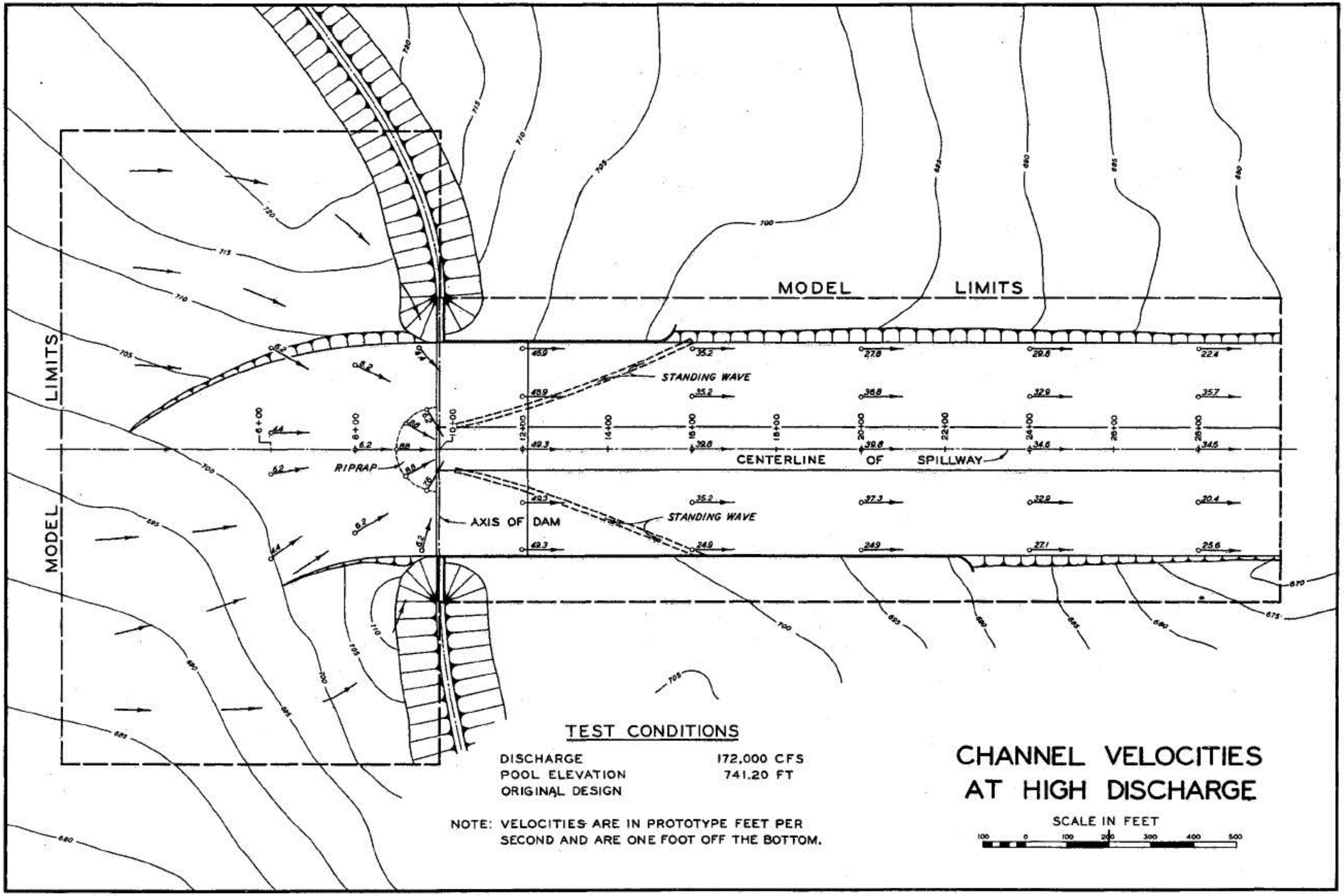
SPILLWAY CREST DETAILS

ORIGINAL DESIGN – TYPE B

PLATE 2

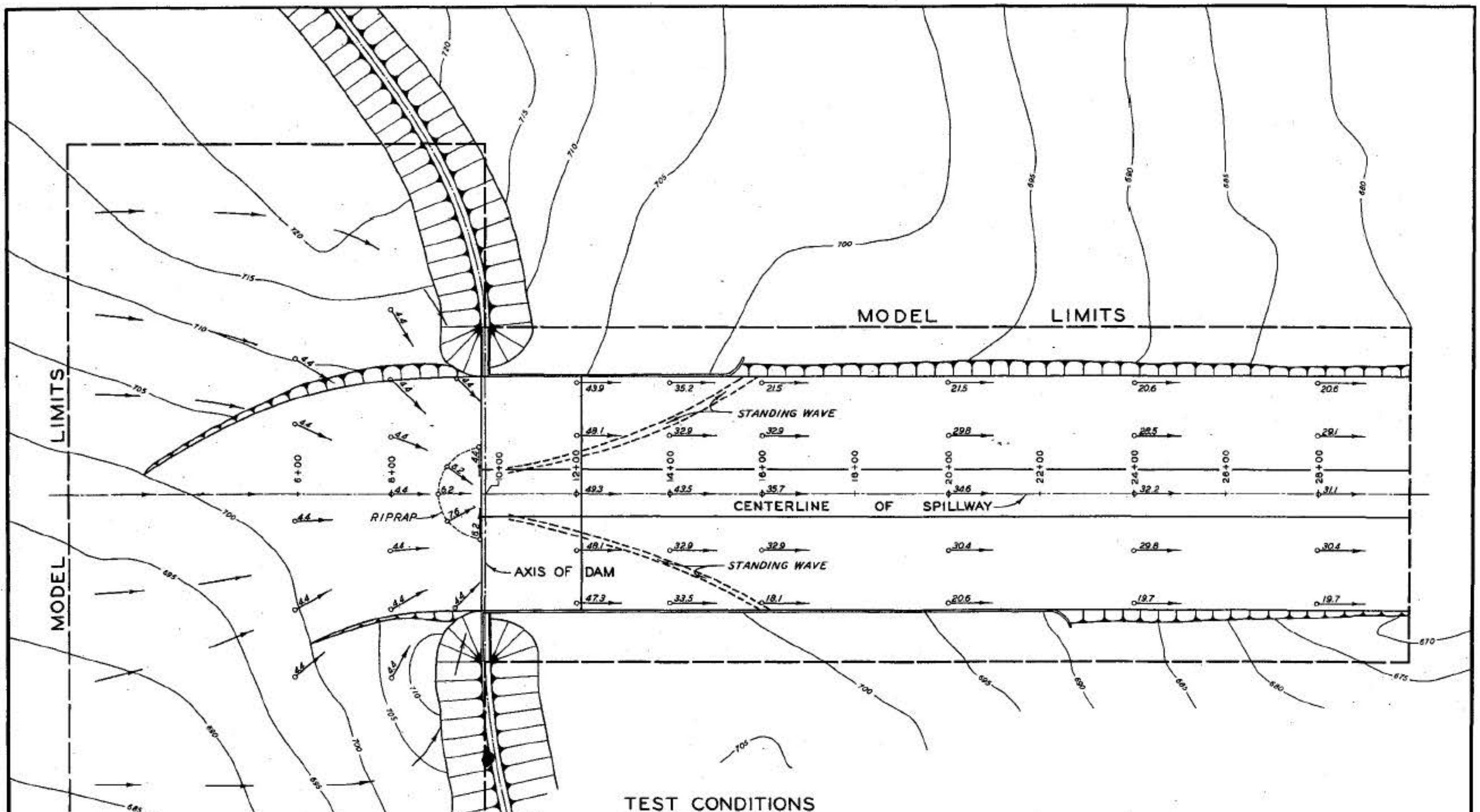


SPILLWAY RATING CURVES



CHANNEL VELOCITIES
AT HIGH DISCHARGE

PLATE 4



TEST CONDITIONS

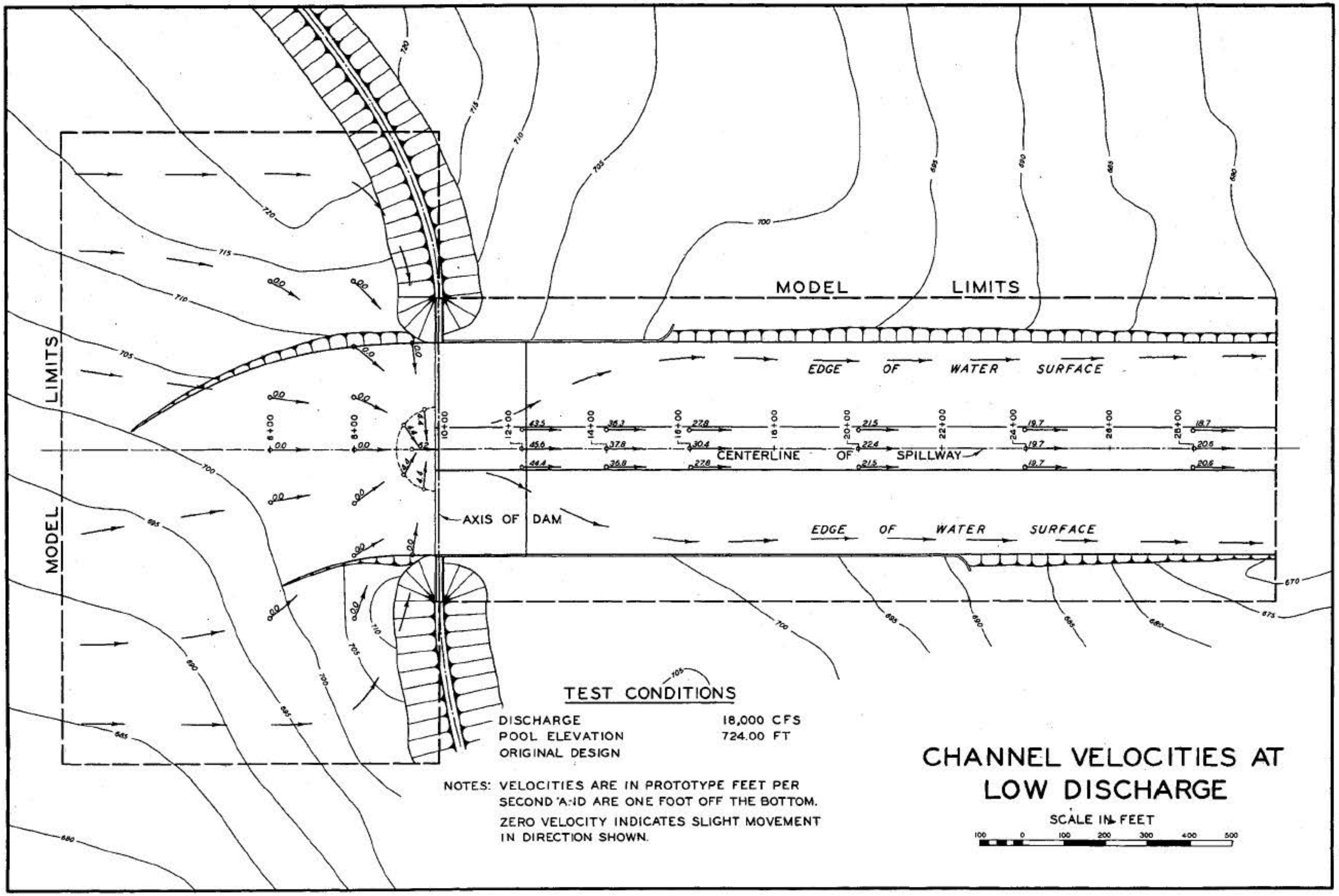
DISCHARGE	100,000 CFS
POOL ELEVATION	735.30 FT
ORIGINAL DESIGN	

NOTE: VELOCITIES ARE IN PROTOTYPE FEET PER SECOND AND ARE ONE FOOT OFF THE BOTTOM.

CHANNEL VELOCITIES AT MEDIUM DISCHARGE

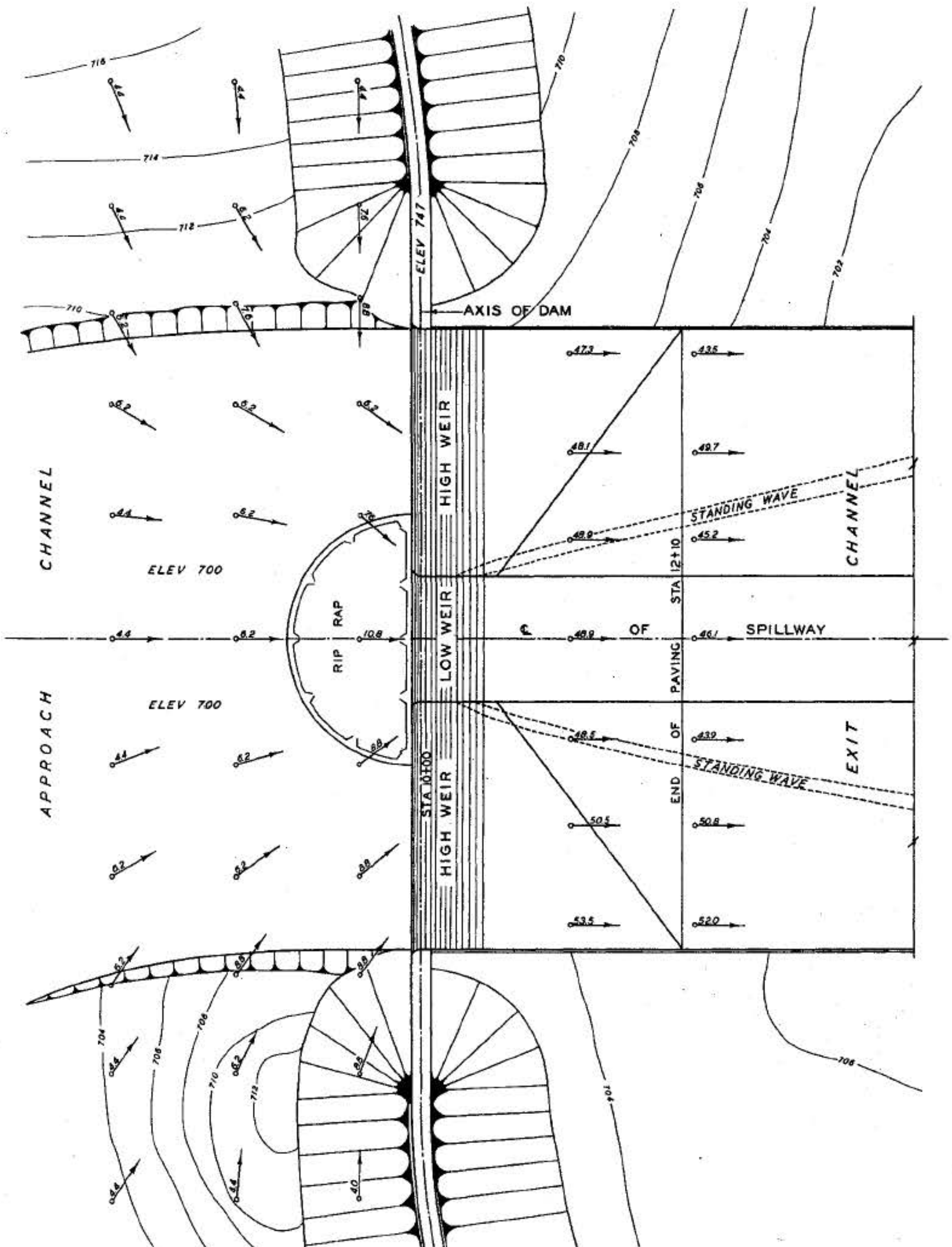
SCALE IN FEET

100 0 100 200 300 400 500



CHANNEL VELOCITIES AT LOW DISCHARGE

PLATE 5



TEST CONDITIONS

DISCHARGE	172,000 CFS
POOL ELEVATION	741.20 FT

NOTE: VELOCITIES ARE IN PROTOTYPE FEET PER SECOND AND ARE ONE FOOT OFF THE BOTTOM.

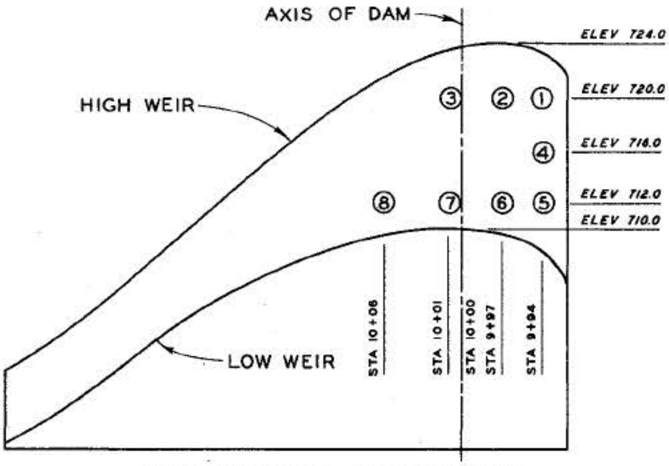
SPILLWAY FLOW PATTERN

ORIGINAL DESIGN

SCALE IN FEET

40 0 40 80 120 160 200

PLATE 6



PIEZOMETER LOCATIONS

PRESSURES *

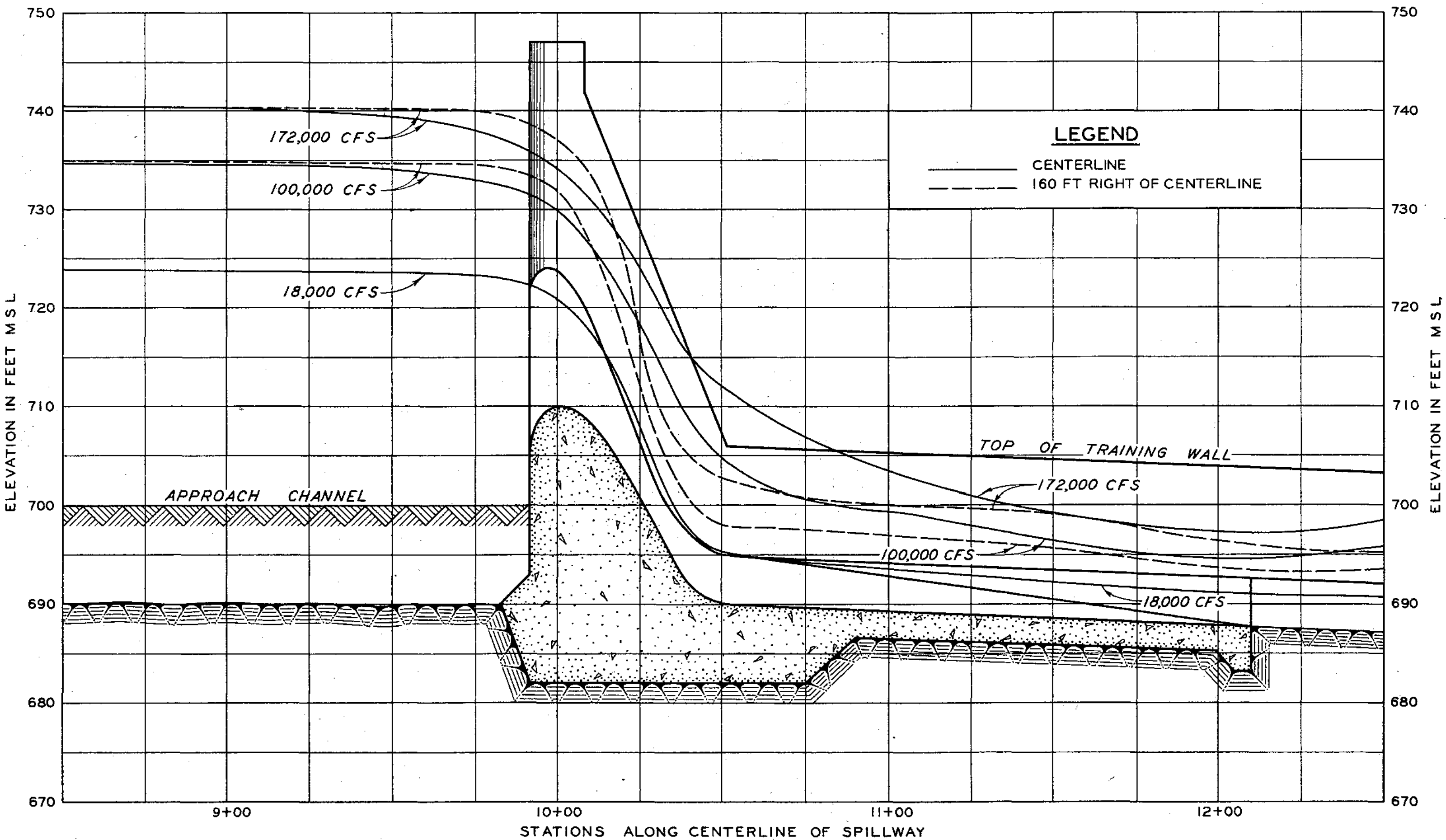
DISCHARGE	172,000 CFS		150,000 CFS		100,000 CFS		80,000 CFS		50,000 CFS	
POOL ELEV	741.2 FT MSL		739.4 FT MSL		735.3 FT MSL		733.2 FT MSL		729.8 FT MSL	
PIEZOMETER NO.	TYPE A	TYPE B	TYPE A	TYPE B	TYPE A	TYPE B	TYPE A	TYPE B	TYPE A	TYPE B
1	-14.0	-5.0	-12.0	-3.0	-8.5	-1.0	-7.5	-1.0	-1.0	
2	8.5	4.5	8.5	4.5	7.5	4.0	6.5	3.5	-1.0	
3	8.5	6.5	8.0	6.0	6.5	5.0	5.5	4.5	0.5	1.0
4	-6.0	-6.0	-4.5	-4.0	-2.0	-0.5	-1.0	0.5	1.0	1.0
5	2.0	-3.0	2.5	-1.0	4.5	2.0	5.0	3.5	4.0	4.5
6	4.0	1.5	4.5	2.0	6.5	4.5	6.5	5.0	4.5	5.5
7	3.0	6.0	4.0	6.5	4.5	7.0	5.0	7.5	7.0	6.5
8	7.0	6.0	7.5	6.5	8.0	6.5	8.0	6.0	6.5	5.0

NOTES: ORIGINAL DESIGN DESIGNATED TYPE A AND TYPE B WHERE:

1. TYPE A CONSISTED OF ORIGINAL DESIGN WITH ABUTMENTS AND LOW WEIR HAVING 90-DEGREE CORNERS.
2. TYPE B CONSISTED OF ORIGINAL DESIGN WITH ABUTMENTS AND LOW WEIR CORNERS ROUNDED.

* PRESSURES ARE IN PROTOTYPE FEET OF WATER.

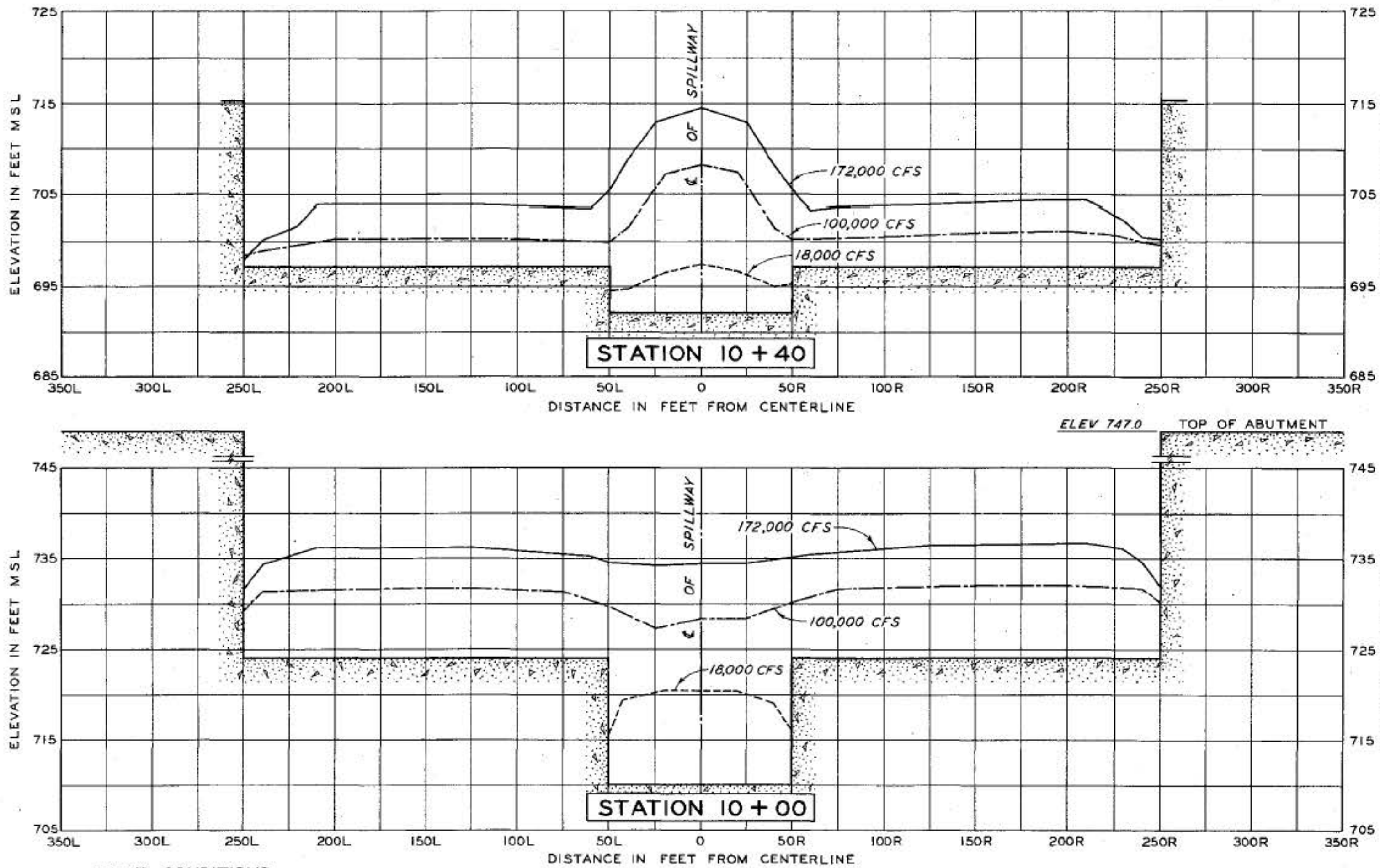
SIDE PRESSURES AT LOW WEIR



TEST CONDITIONS

DISCHARGE	172,000 CFS
POOL ELEVATION	741.20 FT
DISCHARGE	100,000 CFS
POOL ELEVATION	735.30 FT
DISCHARGE	18,000 CFS
POOL ELEVATION	724.00 FT

WATER-SURFACE PROFILES
ORIGINAL DESIGN



TEST CONDITIONS

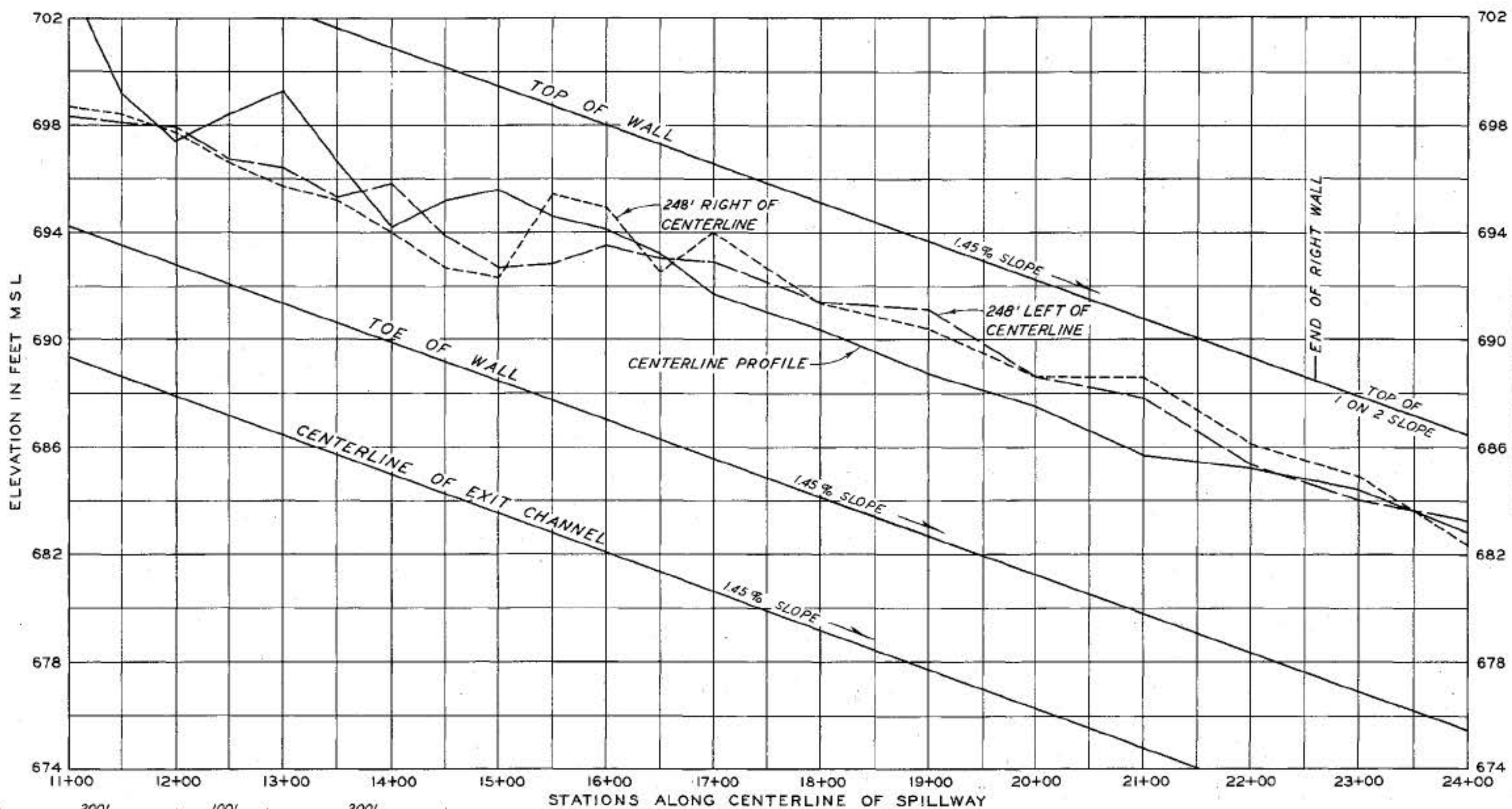
DISCHARGE	172000 CFS
POOL ELEVATION	741.20 FT
DISCHARGE	100000 CFS
POOL ELEVATION	735.30 FT
DISCHARGE	18000 CFS
POOL ELEVATION	724.00 FT

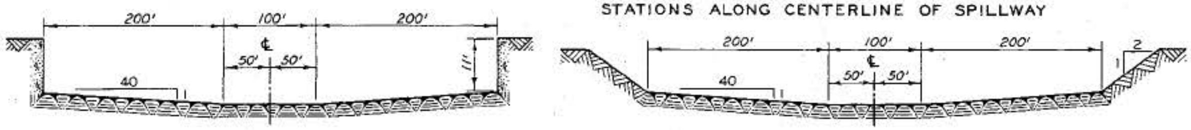
ORIGINAL DESIGN — TYPE B

WATER-SURFACE CROSS SECTION AT SPILLWAY CREST

PLATE 9

PLATE 10





WITH TRAINING WALLS

WITHOUT TRAINING WALLS

TYPICAL CHANNEL SECTIONS

NOT TO SCALE

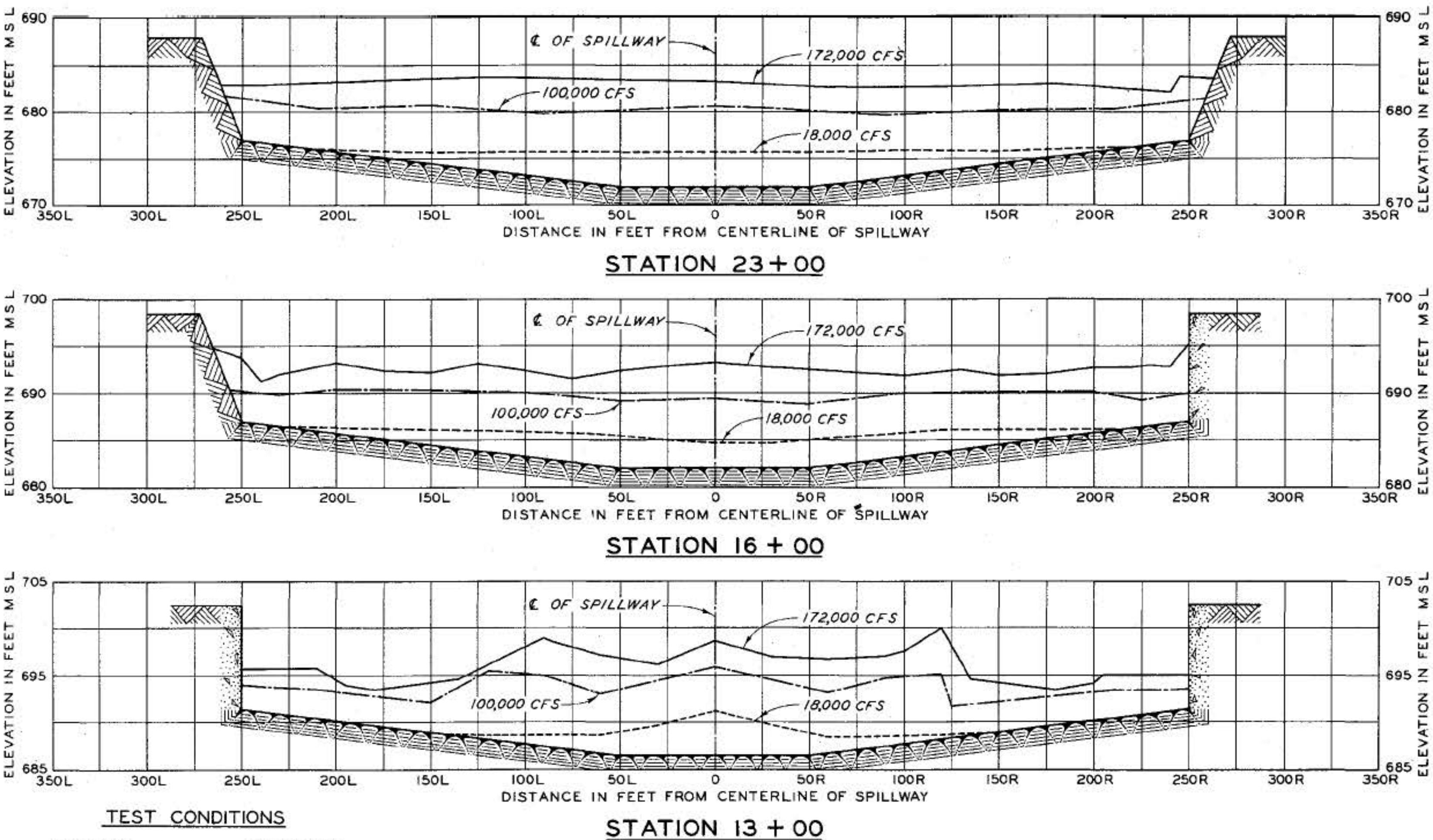
NOTE: TRAINING WALL ENDS AT STATION 15+60 ON LEFT SIDE AND AT STATION 22+60 ON RIGHT SIDE.

TEST CONDITIONS

DISCHARGE	172,000 CFS
POOL ELEVATION	741.20 FT

WATER-SURFACE PROFILES

EXIT CHANNELS (ORIGINAL DESIGN)



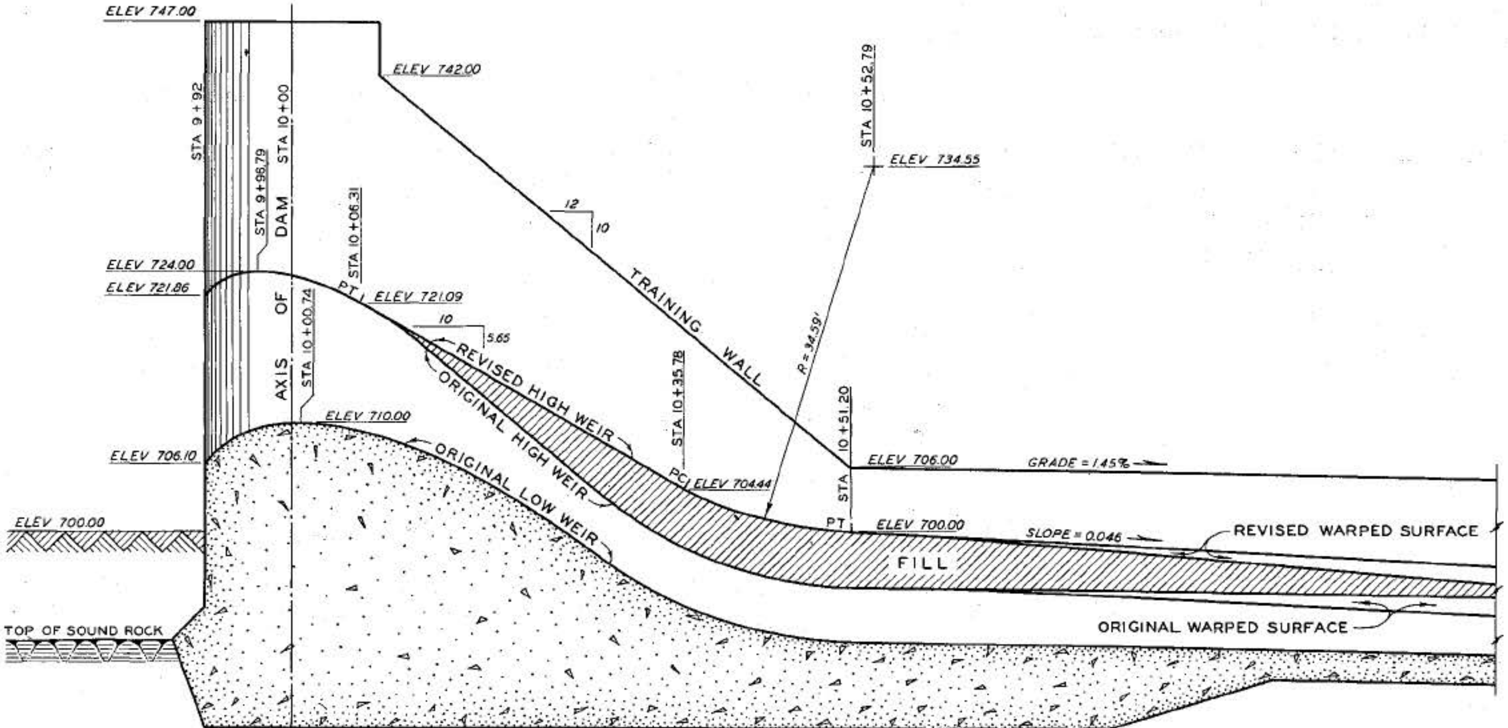
TEST CONDITIONS

DISCHARGE	172 000 CFS
POOL ELEVATION	741.20 FT
DISCHARGE	100 000 CFS
POOL ELEVATION	735.30 FT
DISCHARGE	18 000 CFS
POOL ELEVATION	724.00 FT

ORIGINAL DESIGN — TYPE B

WATER-SURFACE CROSS SECTIONS IN EXIT CHANNEL

PLATE 11

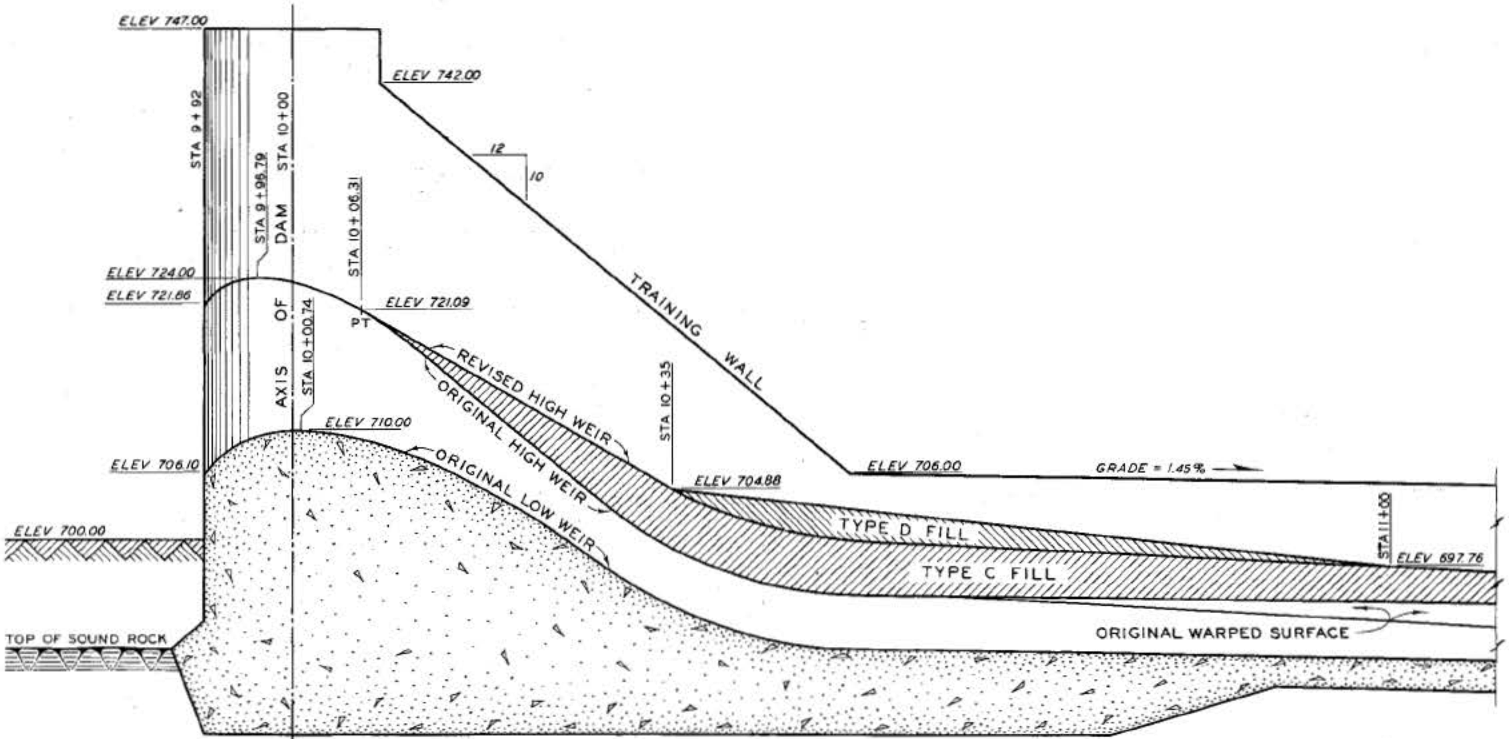


SECTION ALONG CENTERLINE OF SPILLWAY

SPILLWAY REVISION-TYPE C

SCALE IN FEET

5 0 5 10 15 20 25



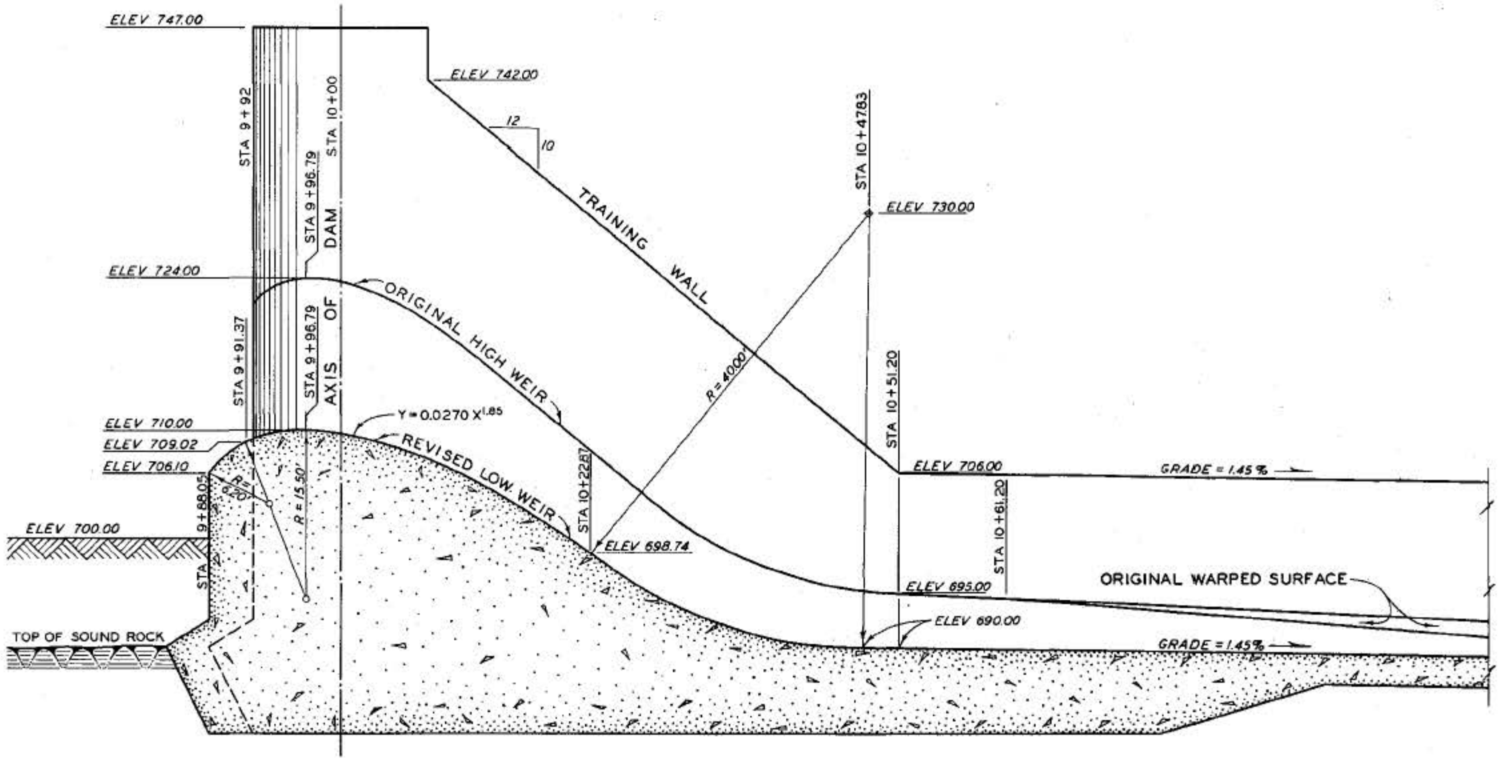
SECTION ALONG CENTERLINE OF SPILLWAY

SPILLWAY REVISION–TYPE D

SCALE IN FEET

5 0 5 10 15 20 25

PLATE 13



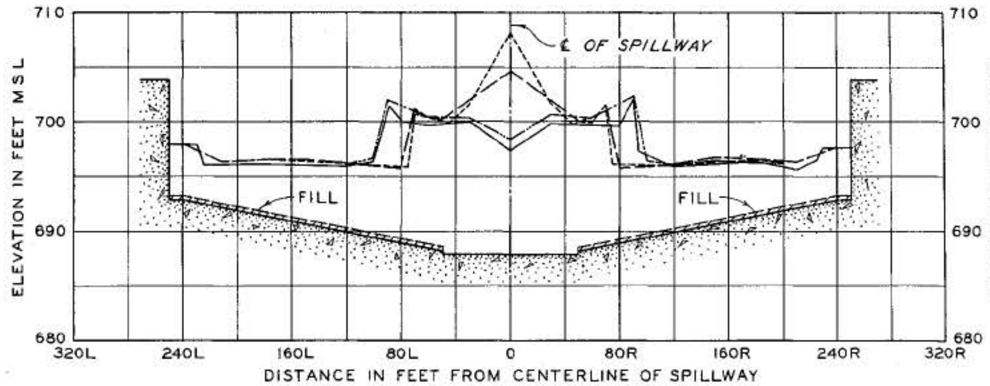
SECTION ALONG CENTERLINE OF SPILLWAY

NOTE: TYPE G ALIGNMENT SAME AS ORIGINAL EXCEPT
LOW WEIR MOVED 3.95 FT UPSTREAM.

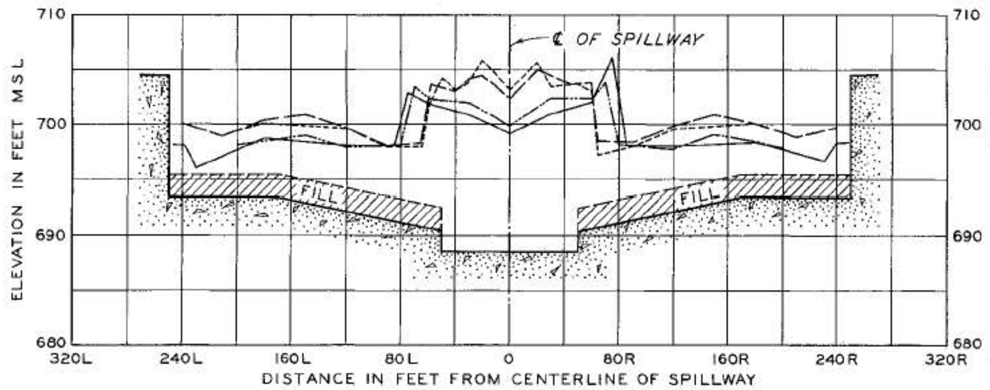
SPILLWAY REVISION–TYPE G

SCALE IN FEET

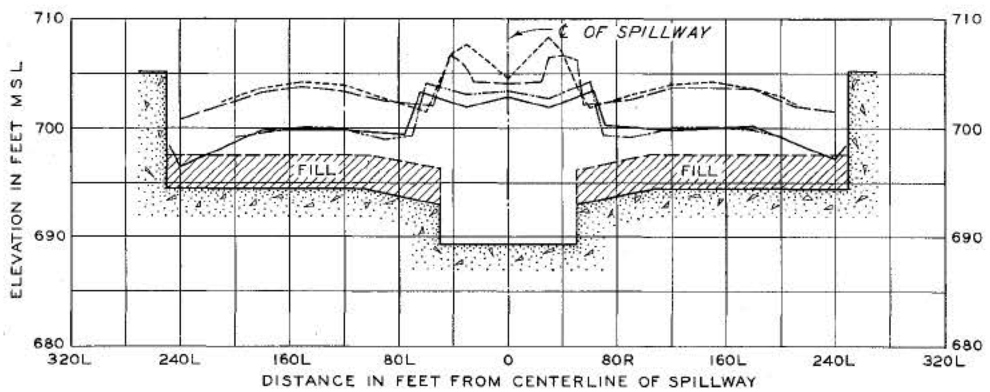
5 0 5 10 15 20 25



STATION 12+00



STATION 11 + 50



STATION 11 +00

LEGEND

- TYPE B (ORIGINAL)
- TYPE C
- TYPE D
- TYPE G

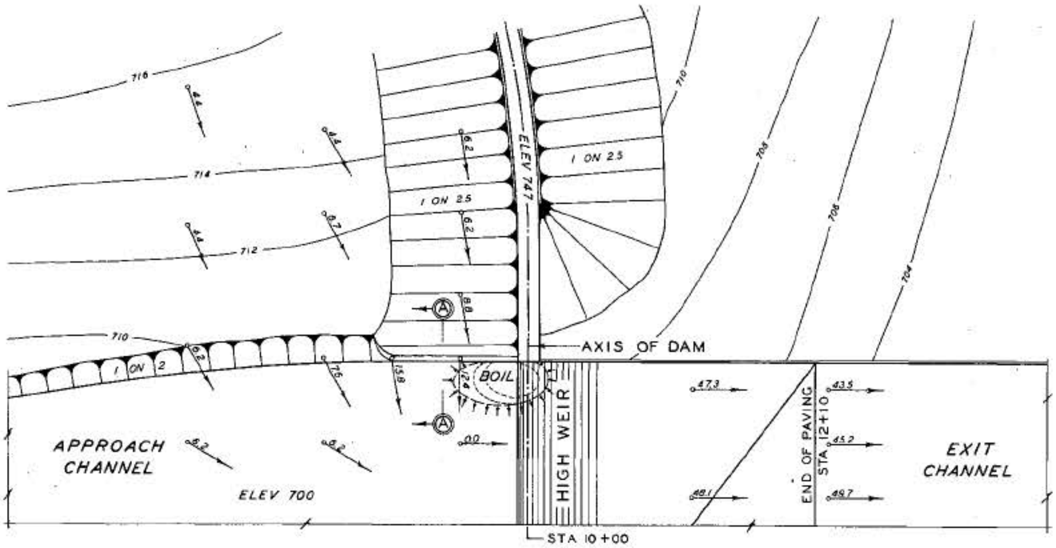
TEST CONDITIONS

DISCHARGE	172,000 CFS
POOL ELEVATION	741.20 FT

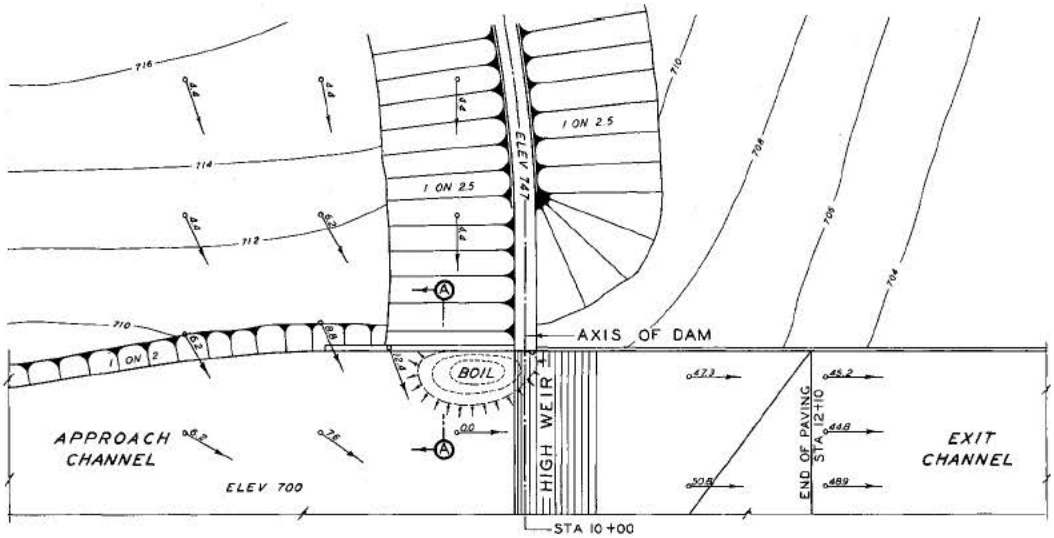
NOTES: FILL SHOWN NOT INSTALLED DURING TESTING OF TYPES B AND G.

ADDITIONAL FILL IN PLACE ABOVE STATION 11+00 FOR TYPE D ONLY.

COMPARATIVE WATER-SURFACE CROSS SECTIONS



1ST ALTERNATE-NO PARAPET

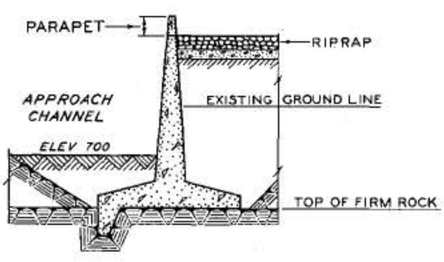


2ND ALTERNATE - 3 FT PARAPET

TEST CONDITIONS

DISCHARGE	172,000 CFS
POOL ELEVATION	741.20 FT

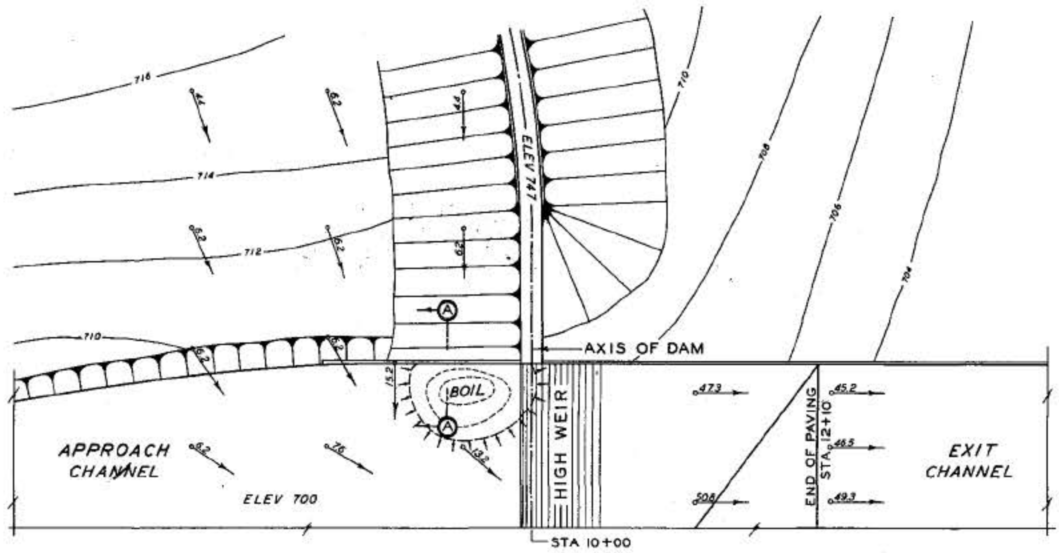
NOTES: VELOCITIES ARE IN PROTOTYPE FEET PER SECOND AND ARE ONE FOOT OFF THE BOTTOM
ZERO VELOCITY INDICATES SLIGHT MOVEMENT IN DIRECTION SHOWN.



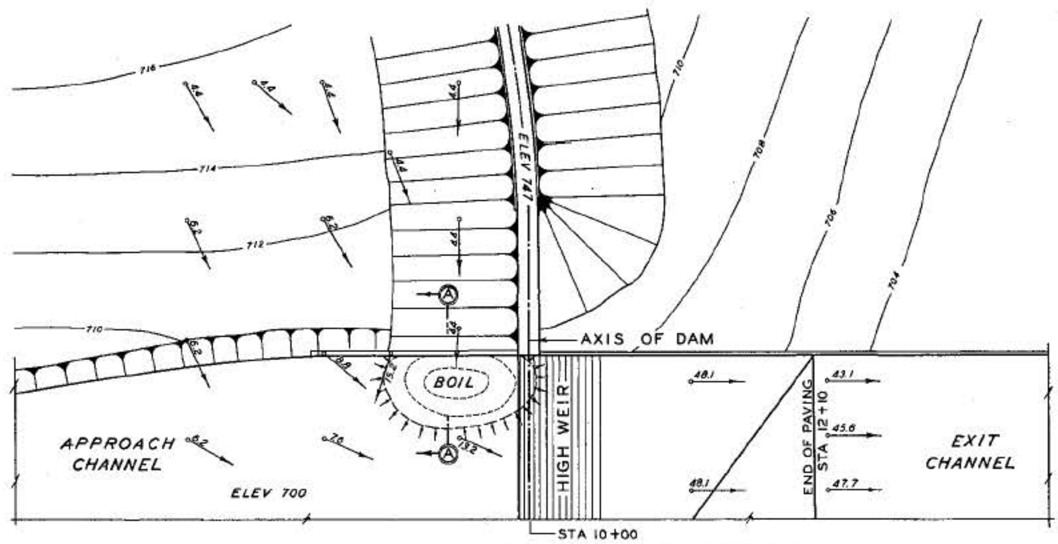
SECTION A-A
(SCHEMATIC)

1ST AND 2ND ALTERNATES
FOR LEFT ABUTMENT

SCALE IN FEET
40 0 40 80 120 160 200



3RD ALTERNATE – 5 FT PARAPET

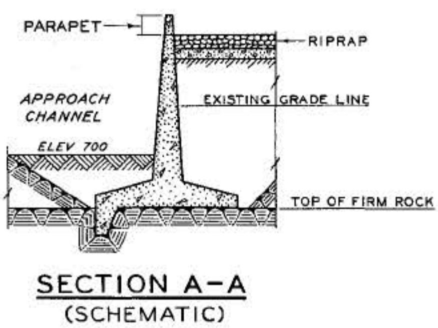


4TH ALTERNATE – 7 FT PARAPET

TEST CONDITIONS

DISCHARGE	172,000 CFS
POOL ELEVATION	741.20 FT

NOTE: VELOCITIES ARE IN PROTOTYPE FEET PER SECOND AND ARE ONE FOOT OFF THE BOTTOM.



SECTION A–A
(SCHEMATIC)

3RD AND 4TH ALTERNATES FOR LEFT ABUTMENT

SCALE IN FEET
40 0 40 80 120 160 200